GEOTECHNICAL INVESTIGATION
EAST WATER PROGRAM, CONTRACT 5C
HOUSTON, TEXAS

Report to

Lockwood, Andrews & Newnam, Inc.
Houston, Texas

by

GEOTEST ENGINEERING, INC.
Houston, Texas

Key Map No. 493R, 494N & P

GEOTEST ENGINEERING, INC.

Materials Testing & Geotechnical Engineers

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Lockwood, Andrews & Newnam, Inc.
1500 CityWest Blvd.
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Attention: Mr. Wayne M. Stevens, P. E.
Program Manager

GEOTECHNICAL INVESTIGATION
EAST WATER PROGRAM, CONTRACT 5C
HOUSTON, TEXAS

Gentlemen:

Presented herein is the report on our geotechnical investigation for the above project. This report contains our recommendations for the proposed water line routes and information on faults and subsidence in the vicinity of the project. This study was authorized by your Task Order No. 026/2(5C) on August 22, 1986.

Preliminary information was provided to you on September 19, 1986.

We appreciate this opportunity to be of service to you. If you have any questions regarding the report, or if we can be of further service to you, please call us.

Very truly yours,
GEOTEST ENGINEERING, INC.

Kuo-Chiang Lin

Kuo-Chiang (Frank) Lin, P.E.
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C O N T E N T S

I L L U S T R A T I O N

A P P E N D I X A

A P P E N D I X B

A P P E N D I X C

INTRODUCTION

The City of Houston has undertaken a capital improvements program involving the analysis and design of improvements to the distribution system of the expanded East Water Purification Plant (EWPP). The design and construction planned for the East Water Purification Plant distribution system improvement includes more than 100 miles of new water lines, pumping stations and pressure reducing valve stations and modifications.

Lockwood, Andrews and Newnam, Inc., (LAN) has been contracted by the City of Houston to provide design engineering and planning for this improvement program. Geotest Engineering, Inc., was retained by Lockwood, Andrews and Newnam, Inc., to conduct the geotechnical engineering studies for this contract.

The primary objectives of this study were to gather information on subsurface conditions at the site and to develop design recommendations for the proposed water lines and to provide information on faults and subsidence. The objectives were accomplished as follows:

1. Drill 11 borings to determine soil stratigraphy and obtain samples for laboratory testing.

2. Install 5 piezometers at selected locations to gain an understanding of the basic groundwater system and to evaluate the potential need for dewatering during construction.

3. Perform laboratory tests to determine physical characteristics of the soils.

4. Perform engineering analyses to develop design guidelines and recommendations for water lines and faults and subsidence in the area.

Subsequent sections of this report contain descriptions of the field exploration and laboratory testing program, area geology, general subsurface conditions and recommendations.

DESCRIPTION OF PROJECT

The scope of study for Contract 5C included a field investigation, a laboratory testing program and an engineering analysis. The purpose of this geotechnical study was to evaluate the subsurface stratigraphy and characteristics of the subsurface soils for the proposed 84 in. (I.D.) water line along Everton from Navigation to Harrisburg and along Harrisburg from Everton to Dowling (about 7,400 feet in length).

Based on the discussions with Lockwood, Andrews and Newnam, Inc. we understand that the proposed water line will be constructed underground using open trench excavation techniques. We also understand that a portion of the 84-in. water line crossing railroad along Harrisburg between Velasco and Roberts may be designed using underground tunneling techniques.

FIELD INVESTIGATION

Subsurface conditions were determined by eleven (11) borings drilled to depths ranging from 25 to 40 feet below the existing grade. One boring drilled by others in the general vicinity of the study area was also utilized. In addition five (5) piezometers, designated as 5C-3P, 5C-5P, 5C-7P, 5C-9P and 5C-11P, were installed at selected locations to monitor groundwater conditions. Boring locations were selected by Geotest and LAN. All the borings at the site were located by Geotest Engineering, Inc. The ground surface elevations at the boring locations were estimated from the City of Houston monumentation maps. The borings and piezometers were drilled with truck-mounted drill rig at the approximate locations shown on Plate 1. Also shown on Plate 1 are the locations of the friction cone and borings drilled by others. Concrete pavement at various boring locations was cored with a 6-in. dia. diamond bit to advance the borings. The thickness of concrete is shown on the respective boring logs.

All boring locations at the site were coordinated with the appropriate authorities to clear the existing underground utilities along the project alignment. A flagman was provided by Geotest Engineering, Inc. during drilling operations to direct traffic on the city streets.

Samples were obtained almost continuously to 10-ft depth and at 5-ft intervals thereafter. Samples of cohesive soils encountered at site were obtained with a 3-in. thin walled tube sampler in general accordance with ASTM Method D 1587-83. Granular soils were sampled with a 2-in. split-barrel sampler in general accordance with ASTM Method D 1586-67. Each sample was removed from the sampler in the field, carefully examined and then classified by an experienced soils technician. Suitable portions of each sample were sealed and packaged for transportation to our laboratory. The shear strength of cohesive soil samples was estimated by pocket penetrometer in the field.

Driving resistances for the split-barrel sampler in granular soils are recorded as "Blows per Foot" on the boring logs. Detailed descriptions of the soils encountered in the borings are given on the boring logs presented on Plates A-1 through A-11 in the Appendix A. A key to soil classification and symbols used on the boring logs is given on Plate A-12 in the Appendix A. The data on friction cone and boring logs provided to us by LAN from study performed by others are presented on Plates A-18 through A-21.

The depth to water in most of the boreholes was measured at various times ranging from one hour to several days after boring completion. The measurement judged to be the best representative groundwater level at each location is recorded in the lower right hand corner of the log.

During our field investigation, no railroad was found at the intersection of Everton and Commerce as it was shown on the City of Houston Monumentation Map. Therefore, no underground tunneling will be required at this location.

INSTALLATION OF PIEZOMETERS

During field investigation piezometers were installed at five (5) selected locations to evaluate the groundwater conditions at this site. Schematic illustrations of all the completed wells are presented on Plates A-13 through A-17 in the Appendix A.

The installation procedure used for all piezometers was basically the same. The wells, about 6-in. diameter, were first drilled to the required depth. A 2-inch PVC with #10 slot screen attached to 2-inch PVC riser pipe was lowered into the hole. The screen and riser pipe were centered in the hole and filter sand was placed around the screen to a desired level above the top of the screen. The depth to the sand pack was checked with a steel tape. Bentonite pellets were dropped around the riser pipe to form a bentonite seal of about 2 feet. Bentonite pellets were dropped individually to prevent honeycombing in the hole. Depth to bentonite seal was checked with a steel tape. The remainder of the annular space between the hole and riser pipe was filled with cement grout to the ground surface. The PVC riser pipes were cut and capped to limit rain water infiltration.

DEVELOPMENT OF PIEZOMETERS

All piezometers were developed using an air compressor with maximum capacity of 250 psi pressure and flexible high-pressure air hose. During development, the air line was placed such that its lower end was a few feet above the bottom of the screen. The air valve was then closed to allow pressure to build up. The valve was then quickly opened to surge the water outward through the well screen and sand pack. This operation was repeated until water was observed free of sand. The air line was then raised to a position of a few feet higher and the same operation was repeated until the entire screen was developed.

OBSERVED GROUNDWATER LEVEL

The groundwater level observations were made in the borings and in the piezometers during the course of this study. The observed groundwater levels at the piezometer locations are given below:

DATE OF	DEPTH BELOW GRADE IN FEET					ELEVATION IN FEET				
READING	5C-3P	5C-5P	5C-7P	5C-9P	5C-11P	5C-3P	5C-5P	5C-7P	5C-9P	5C-11P
9-04-86	8.0	18.0	12.5	12.5	13.0	20.0	24.0	27.7	27.5	29.0
9-09-86	7.3	8.5	13.3	6.7	9.2	20.7	33.5	26.9	33.3	32.8
9-17-86	6.2	9.1	12.0	5.9	9.0	21.8	32.9	28.2	34.1	33.0

As can be seen from the above data, the groundwater level during the field investigation ranged from 6 feet (El. 34') to 12 feet (El. 22') below ground surface.

LABORATORY TESTS

The laboratory testing program was directed primarily towards evaluation of the pertinent physical properties and shear strength characteristics of the foundation soils. Classification tests were performed on selected samples to aid soil classification.

Undrained shear strengths of selected cohesive samples were determined by unconfined compression (UC) tests and unconsolidated-undrained (UU) triaxial compression tests. The results of unconfined compression and UU triaxial tests are plotted on the boring logs as solid circles and squares, respectively. Stress-strain curves were developed for the UU triaxial tests to estimate the modulus of elasticity. The stress-strain curves for the UU triaxial tests are presented on Plates B-1 through B-7 in Appendix B. Shear strengths of cohesive samples were also determined in the field with a calibrated hand penetrometer and in the laboratory with a torvane. The results of these values are plotted on the boring logs as open circles and triangles, respectively. Shear strength values found to be greater than 1.5 TSF, as determined by penetrometer, are shown on the boring logs as O^{+}.

Water content and unit dry weight of the foundation soils were determined as a part of the unconfined compression and UU triaxial tests. Water content determinations were also made on other samples to define the moisture profile at each boring location. Liquid and Plastic Limit tests were performed on appropriate cohesive samples. Percent passing No. 200 sieve tests were also performed on appropriate samples to aid soil classification. The results of all tests are plotted or summarized on the boring logs. Sieve analyses were also performed on selected granular samples. The results of sieve analyses are presented in the form of grain size distribution curves on Plate B-8 in the Appendix B.

The type and the number of tests performed for this study are listed below.

TYPE OF TEST	NUMBER OF TEST
Hand Penetrometer	92
Water Content (ASTM D2216)	75
Liquid Limit (ASTM D4318)	43
Plastic Limit and Plasticity Index (ASTM D4318)	43
Unconfined Compressive Strength of Cohesive Soils (ASTM D2166)	18
Torvane	27
Unconsolidated-Undrained Triaxial Test (ASTM D2850)	7
Stress-Strain Curves	7
Particle Size Analysis (ASTM D422)	3
Standard Penetration Test (ASTM D1586)	2

AREA GEOLOGY

Geologic conditions in the project area were evaluated by reviewing published geologic, faulting, and subsidence information.

The project area is located within the Texas Coastal Zone. The project is underlain by the Beaumont formation, which is of Pleistocene age (25,000 to 2,000,000 years before present). Beaumont soils consist of clays, with interbedded silts and sands, deposited in river deltas and floodplains. Subsequent to their deposition, the soils were desiccated when sea level was much lower than it is now. As a result of these geologic processes, the upper Beaumont soils are overconsolidated, have moderate-to-high strength and low compressibility within the range of moderate foundation pressures.

The project is located in a Seismic Zone 0 according to the Uniform Building Code, which indicates that there is essentially no risk of earthquake damage. The primary hazards in the Texas Coastal Zone are subsidence (and related flooding) and surface faulting, both of which are affected by groundwater withdrawal from deep wells.

The stratigraphic units that outcrop and are present in the near subsurface along the alignment of Contract 5C are the Pleistocene Beaumont Formation and man-made fill. The Beaumont Formation consists of stiff to hard, low to high plasticity clays, medium dense silts to clayey silts, and dense to very dense silty sands. The depositional environment of the Beaumont Formation in this area was on the lower alluvial-deltaic plain of the ancestral Brazos River.

Borings along the alignment were terminated in low to high plasticity clays which most likely represent overbank floodbasin deposits. These Beaumont Formation sediments were deposited approximately 50,000 to 40,000 years ago in this area.

GENERAL SUBSURFACE CONDITIONS

The subsurface conditions along the proposed alignment for Contract 5C were determined by thirteen borings, borings 5C-1 through 5C-11, 5B-13 and 5D-5 and one friction cone, C-1. Based on the subsurface soils revealed by the test borings, two generalized soil profiles have been developed to evaluate the subsurface conditions; one along the general alignment of Harrisburg and one along the alignment of Everton. The generalized soil profiles are presented on Plates 2 and 3. Minor textural and color variations and inclusions of other materials within each generalized stratum are shown on the boring logs (Plates A-1 through A-11 and A-21). It should be noted that the observed groundwater level may fluctuate seasonally due to climatic conditions. The subsurface soils along each street are described below.

Harrisburg

Based on the soil conditions revealed by test borings, borings 5C-1 through 5C-7, and 5D-5 and friction cone C-1, a generalized soil profile was developed along the alignment and is presented on Plate 2. In general, the surficial soil consists of clay and sandy clay fill to depths ranging from 4 to 6.5 feet. The fill is of medium to high plasticity with liquid limit ranging from 36 to 75 percent and plasticity index ranging from 19 to 47 percent. The natural moisture content of the fill ranged from 9 to 23 percent. The shear strength of the fill ranged from 1000 to 3000 + psf. Below the fill, alternating layer of stiff to very stiff tan and light gray clay and sandy/silty clay were encountered to a depth of 40 feet (El. -12'), the maximum depth explored. The clays are of low to high plasticity with liquid limit ranging from 20 to 67 percent and plasticity index ranging from 8 to 41 percent. The natural moisture content of the clays ranged from 13 to 29 percent and unit dry weight ranged from 100 to 122 pcf. The shear strength of clays was found to vary from 1000 to 4000 psf. Secant modulus at 50 percent of the maximum deviator stress from the unconsolidated-undrained triaxial test was found to range from 104 to 292 ksf. In boring 5C-3, a stratum of medium dense light gray silty fine sand was encountered at 7-foot depth (El. 21'). The natural moisture content of this sand layer was 22 percent. The SPT value obtained from standard penetration test was 21 blows/ft. The percentage of the granular material passing a No. 200 sieve was about 39. In boring 5C-7, medium dense tan silty fine sand was encountered at 36-foot depth (El. 4'). The natural moisture content of this sand layer was about 20 percent and the SPT (Standard Penetration Resistance) value was about 27 blows/ft. The granular material has 18 percent finer than No. 200 sieve.

Groundwater level was found to range from about 6 feet (El. 33') to 14.5 feet (El. 22') below ground surface.

Everton

Based on the soil conditions revealed by test borings, borings 5C-7 through 5C-11 and 5B-13, a generalized soil profile was developed along this alignment and is presented on Plate 3. The surficial soil consists of clay fill of high plasticity. The thickness of the fill varies from 4 to 6.5 feet. The natural moisture content of the fill ranged from 3 to 33 percent. The liquid limit of the fill ranged from 62 to 66 percent and plasticity index ranged from 38 to 41 percent. The shear strength of the fill was found to vary from 1200 to 2400 psf. Below the fill, alternating stratum of stiff to very stiff tan and light gray clay and sandy clay were encountered to a depth of 36 feet (El. 4'). The clays are of medium to high plasticity with liquid limit ranging from 22 to 81 percent and plasticity index 9 to 52 percent. The natural moisture content of the clays ranged from 13 to 32 percent and the unit dry weight ranged from 96 to 121 pcf. The shear strength of the clays was ranging from 1200 to 3000 $^{+}$ psf. Secant modules at 50 percent of the maximum deviator stress from the unconsolidated-undrained triaxial test ranged from 104 to 530 ksf. In boring 5C-7, medium dense tan silty fine sand was encountered to a depth of 40 feet (El. 0'), the maximum depth explored. The natural moisture content of this sand layer was about 20 percent. The SPT (Standard Penetration Resistance) value was about 27 blows/ft. The percentage of the granular material passing a No. 200 Sieve was about 18.

Groundwater level was found to range from about 6 feet (El. 34') to 12 feet (El. 28') below ground surface.

RECOMMENDATIONS

Pipe Line Excavation

It is our understanding that the proposed water line for Contract 5C will be constructed underground, using trench excavation techniques. A portion of the proposed line crossing railroad along Harrisburg between Velasco and Roberts may be designed using underground tunneling techniques. The excavation will require certain dewatering procedures, which are discussed in a separate section in this report.

The excavation for Eastside Water Distribution System will extend to the edges of the property lines or adjacent to other sites on which structures already exist. Also the alignment is running along the City streets and the traffic flow must be maintained. Under these circumstances, we expect that braced excavation will be the most appropriate method of construction. The sides of the excavation should be made vertical and should be supported to provide safety for workers and adjacent structures. Techniques requiring the pulling of a steel box may be considered.

The excavation for the 84-in. water line is anticipated to range from about 15 ft. to 19 ft. below the existing ground surface. For such conditions, steel sheet piles are commonly used and are driven along the line of the excavation before the soil is removed. As the excavation proceeds, wales and struts are inserted to minimize lateral movement of the sheeting. For excavation in stiff clays, several square feet of vertical face of wall excavation can be exposed without collapsing. It may then be possible to replace steel sheet piles with a series of driven H-piles spaced at 4 to 6 feet. As the soil next to the piles is removed, wood laggings are installed and are wedged against the soil outside the cut. As the excavation advances from one level to another, wales and struts are inserted in the same manner as for the steel sheeting. Typical details of bracing systems for deep excavations are presented on Plate 4.

In braced cuts, if sheeting is terminated at the base of the cut, the bottom of the excavation can become unstable under a certain condition. This condition is governed mainly by the differential hydrostatic head. In cohesionless soils, if encountered, excavation should be made after dewatering is accomplished and consequently no bottom stability problem is anticipated. For cuts in clays, stability of the bottom can be evaluated in accordance with the procedure outlined on Plate 5. For the proposed 19-ft. excavation, the calculated factor of safety in clays against bottom stability was found to be in excess of 4.0.

Lateral Earth Pressure on Braced Excavation

For design of braced excavation a thorough knowledge of the exact nature of the materials through which excavation is to be performed is essential. An estimate was made of the soil parameters such as total unit weight of the soil, angle of internal friction of granular soils and shear strength of the clays. Lateral earth pressure was computed for the 15-ft. and 19-ft. excavations. The computed values and the pressure distribution are presented on Plate 6 in a graphical form. For our engineering analysis a minimum backfill cover of 6 ft and a maximum cover of 10 ft has been assumed. The groundwater level was assumed to be 6 ft below ground surface. The dewatering procedures will be discussed in the section of "Dewatering Requirements" in this report.

Steel struts should be designed for the compressive stress allowed by the customary column formulas; and wood struts should be designed with two-thirds the customary compressive stress. Struts must be carefully cross-braced to prevent damage from impact of construction equipment.

Bracing design should consider Occupational Safety and Health Administration Standards as minimum for design.

Vertical Loads on Ditch Conduits

Vertical Earth Pressure on Conduits. The vertical load on an underground conduit depends principally on the weight of the prism of soil directly above it. Also, the load is affected by vertical shearing forces along the sides of the prism caused by differential settlement of the prism and adjoining soil; the shearing forces may be directed up or down. This relative movement along the sides of the ditch mobilizes certain shearing stresses or friction forces which act upward in direction and which, in association with horizontal forces, create an arching action that partially supports the soil backfill. Hence, the load on the conduit may be less or greater than the weight of the soil prism directly above it. The difference between the weight of the backfill and these upward shearing stresses is the load that must be supported by the conduit at the bottom of the ditch. A free-body diagram of the vertical pressure acting on the top of the conduit is presented on Plate 7.

For a rigid conduit (e.g., prestressed concrete embedded cylinder pipe or steel pipe), vertical load due to overburden can be estimated using the following equation (Spangler, 1982):

$$W_c = C_d \, r \, B_d^2 \qquad (1)$$

in which

W_c = vertical load per unit length of conduit, in lbs/linear ft

C_d = load coefficient.

r = wet unit weight of backfill material (recommended 125 pcf); and

B_d = Horizontal width of trench at top of conduit, in feet

The load coefficient C_d is a function of the trench depth to width ratio and the frictional characteristics of the backfill material and sides of the trench. Values of C_d for use in design should be obtained from Plate 8. Sample calculations and computed values of W_c for various widths are presented on Plate C-1 in Appendix C.

<u>Load on Conduit Due to Traffic Loads</u>. In addition to the vertical earth pressure or overburden, underground conduits are also subject to live loads, such as wheel loads applied at the surface of the backfill and transmitted through the soil to the underground structure. The live load on conduit due to traffic loads can be calculated using the following equation (Spangler, 1982):

$$W_t = \frac{1}{A} I_c C_t P \quad \text{.........................} \quad (2)$$

in which

W_t = average load per unit length of conduit due to wheel load;

A = effective length of conduit section on which load is computed;

I_c = impact factor;

C_t = load coefficient; and

P = concentrated wheel load on surface

The impact factor I_c is a function of vehicle speed, its vibratory action, and the roughness characteristic of the roadway surface. For a stationary vehicle, I_c is equal to unity. For trucks operating on an unpaved roadway, I_c may be taken as 1.5 (Spangler, 1982). The load coefficient C_t is dependent on the length and width of the conduit section and depth of cover and may be evaluated according to the Boussinesq solution for stress distribution. To estimate the live load on a circular or arch-shaped conduit it is valid to calculate the load on a rectangular area which is the vertical projection of the conduit section on a horizontal plane through the top of the structure. The value of effective length A should be taken as 3 ft for conduits greater than 3 ft in length and the actual length for conduits less than 3 ft in length.

For the 3 ft effective length of the pipe (84-in. I.D.) the value of C_t was computed to be 0.230 for 6-ft. cover and 0.102 for 10-ft. cover.

Sample calculations and computed values of W_t for various depths and pipe I.D. are presented on Plate C-2 in Appendix C.

Pipe Bedding and Trench Backfilling

The requirements for pipe bedding, encasement materials and compaction are presented on Plate 9.

Regardless of the type of pipe being laid, 6" of sand bedding shall be provided at the bottom of the trench prior to laying the pipe and making up the joints. Subsequent to completion of joints being made up and inspected, sand backfill shall be placed around the pipe, extending the full width of the trench and to a minimum compacted depth of 6" over the top of the pipe to provide a compacted encasement surrounding the pipe. Care shall be taken to see that no dirt, clods or trench sides are allowed to fall and/or to rest against the pipe prior to the completion of the sand encasement.

Sand for bedding and encasement shall be a select sandy soil or other granular material being free from clay lumps, organic materials or other deleterious substances and having a plasticity index of not greater than 7 and with not more than 40 percent passing a No. 200 sieve.

The trench shall then be backfilled in accordance with specifications provided on Plate 9.

The settlement of water lines, bedded, encased and backfilled in accordance with the above specifications should be negligible.

<u>Three Edge Bearing Load</u>

In order to design a rigid conduit, the vertical load due to overburden and live load due to traffic, as determined by equations 1 and 2, must be modified by a load factor which includes bedding conditions and relates the maximum load on the conduit to the three-edge bearing load which causes a crack 0.01 inch wide in a test specimen of the conduit. This modified load is commonly referred to as "D-load" or "Three Edge Bearing Load". The three edge bearing load can be determined from the following equation:

$$S_{eb} = \frac{(W_c + W_t) \times F_s}{L_f} \qquad \text{...............} \quad (3)$$

S_{eb} = Three edge bearing load in lbs/lin ft

W_c, W_t = Maximum Load on conduits in lbs/lin ft from overburden and traffic, respectively

F_s = Factor of Safety = 1.2

L_f = Load factor = 1.5 for bedding conditions recommended on Plate 9

Sample calculations for Three Edge Bearing Load are presented on Plate C-3 in Appendix C.

<u>Dewatering Requirements</u>

Along Harrisburg and Everton Street (see soil profiles on Plates 2 & 3), the groundwater level was encountered between 6 and 14.5 ft. below existing ground surface. The excavations along the Contract 5C alignment are expected to range from 15 to

19 feet below existing grade. For this segment a conventional pump and sump arrangement is considered adequate, except in the vicinity of HB and T railroad bridge and the intersection of Harrisburg (West Bound) and Dowling. At these locations well points may be required to lower the groundwater level to at least 3 feet below the excavation level.

Piping System Thrust Restraint

Forces Due to Pipe Bends. Unbalanced thrust forces will be developed at changes in pipe direction due to reaction of the force producing this stress in the pipe. In all bends, there will be a slight loss of head due to turbulence friction. This loss will cause a pressure change across the bend, but it is usually small enough to be neglected.

The force diagram shown on Plate 10 illustrates the trust force generated by flow at a bend in the pipe. The equations for computing this thrust force are also given on this Plate. The values of thrust forces for a surge pressure of 210 psi were computed for various bend angles and these values are presented on Plate C-4 in Appendix C.

The thrust forces generated at the locations having a horizontal bend along Contract 5C alignment are given below:

LOCATION	PIPE I.D.	BEND ANGLE	SURGE PRESSURE	THRUST FORCE
Harrisburg X Dowling	84"	90°	210 psi	1646 kips
Harrisburg X HB & T R.R. (near St. Charles)	84"	38°	210 psi	758 kips
Along Harrisburg between Palmer Velasco	84"	10°	210 psi	203 kips
Harrisburg X Everton	84"	90°	210 psi	1646 kips
Everton X Navigation	84"	73°	210 psi	1385 kips

The thrust force will require more reaction than is available just from the pipe bearing against the backfill. In order to prevent intolerable movement and overstressing of the pipe, suitable buttressing should be provided. In general, the thrust blocks, restrained joints and tie rods are common methods of providing reaction for the thrust restraint design and will be discussed in following sections.

Bearing Thrust Block. A typical bearing thrust block arrangement for a horizontal bend is shown on Plate 11. The free body diagram of the thrust and reaction forces along with the design equations are also given on this Plate. The bearing block should be placed against undisturbed soil. Usually the block height (h) should be equal to or less than one half the total depth to the block base (H_T), and also should not be less than the conduit outside diameter. In general, the block width (b) varies from one to two times the block height (h).

Based on the soil conditions revealed by the borings, the thrust block is likely to be located entirely in clay. For such conditions the following soil parameters are recommended for the design of the thrust blocks.

Soil internal friction angle,	ϕ	=	0^o
Passive earth pressure coefficient,	K_p	=	1
Soil unit weight,	r	=	125 pcf (0' - 6')
	r'	=	62.5 pcf (below 6')
Soil cohesion,	C	=	2000 psf

Computations were made to size the thrust block for various soil covers. The computed dimensions of thrust blocks are presented on Plate C-5 in Appendix C.

Our analysis indicates that the bearing thrust block is not feasible at the locations where the 84-in. I.D. pipe has a horizontal bend greater than 23 degrees for 6 ft. cover and 32 degrees for 10 ft. cover. At these locations the dimension of the bearing thrust block cannot be rationally adjusted for the corresponding depth and height of the block.

Gravity Thrust Block. A typical gravity thrust block and the design equation are presented on Plate 12. The horizontal thrust component (T_x) is counteracted by soil pressure on the vertical face of the block (F_p) or by joint restraint. The minimum size of the block base can be determined by allowable soil bearing pressure.

At the intersection of Harrisburg and HB&T Railroad the vertical bend of the pipe may be utilized and gravity thrust block will be needed. Based on the soil conditions revealed by borings 5C-2 and 5C-3 the soil parameters for designing the gravity thrust block at this location are summarized below:

Soil internal friction angle, $\emptyset = 0^o$

Passive earth pressure coefficient, $K_p = 1$

Soil unit weight, r = 125 pcf (0' - 6')
r'= 62.5 pcf (below 6')

Soil cohesion, C = 2000 psf

Allowable bearing pressure, q_a = 6000 psf

Restrained Joints. Occasionally, thrust blocks are not a practical or economical reaction system due to limited space, access, unstable soils, or possible disturbance due to future excavations. Where thrust blocks are not practical, restrained joints, allowing thrust and shear forces to be transmitted across the pipe joints, are employed to allow a number of pipe sections to act integrally in bearing. A plan view and free body diagram of a pipe bend employing the pipe with restrained joints to provide reaction to the thrust is given on Plate 13. The equation necessary to determine the length of joint restrained pipe is also provided on this Plate. It can be seen from the equation presented on Plate 13 that the unit weight, cohesion and/or angle of internal friction of the soil are required for a rational design.

The in-situ soil parameters presented above under the section of Bearing Thrust Block can be used to determine the lateral soil resistance (passive soil pressure) at the bend locations along Contract 5C alignment. The backfill parameters presented on Plate C-6a in the Appendix C should be used to determine the sliding frictional resistance on the pipe. Based on the size of the pipe, the depth of the cover and the soil conditions encountered at the site, the normal force (W) exerted on the pipe by the surrounding backfill should be computed using the average pressure distribution method. The force, W is equal to $\pi r_b H B_c R$, where r_b is the unit weight of the backfill and H is the depth of the cover, B_c is the outside diameter of the pipe and R is the reduction factor depending on the compaction of backfill and generally is equal to 2/3.

The estimated length of joint restrained pipe at the bend locations along Contract 5C alignment were estimated and are tabulated below.

LOCATION	PIPE I.D.	BEND ANGLE	SURGE PRESSURE	RESTRAINED PIPE LENGTH 6 ft Cover	10 ft Cover
Harrisburg X Dowling	84"	90°	210 psi	103 ft.	91 ft.
Harrisburg X HB & T R.R. (near St. Charles)	84"	38°	210 psi	44 ft.	40 ft.
Along Harrisburg between Palmer & Velasco	84"	10°	210 psi	12 ft.	11.5 ft.
Harrisburg X Everton	84"	90°	210 psi	103 ft.	91 ft.
Everton X Navigation	84"	73°	210 psi	82 ft.	74 ft.

Sample calculations for determining the required length of joint restrained pipes are presented on Plate C-6 in Appendix C.

Tie Rods. The unbalanced thrust forces can also be achieved by using tie rod system such as anchorage to structure, thrust collars, deadman anchors, joint restraint by utilizing clamps and pipe flange. By considering the soil friction and lateral soil resistance, the effective thrust force at a joint (T_j) can be related to its distance from bend (L_j) and the restrained length (L) as shown on Plate 14. The equations required for computing the thrust force and the number of tie rods are also presented on Plate 14.

The soil (backfill) parameters for designing the tie rods are summarized below:

Soil internal friction angle, Ø, degrees	25
Passive earth pressure coefficient, K_p	2.46
Soil unit weight, r, pcf	120
Soil cohesion, C, psf	0

When geometric restriction, excessive thrust forces, soil conditions or economics prevent utilization of one or a combination of these methods of thrust reaction, it is not uncommon to gain the additional reaction with batter piles. Consequently, throughout the distribution system, all locations where thrust reaction will be required should be evaluated with respect to potential reaction systems. Development of these conceptual reaction systems will dictate the requirements for soil data at these locations.

Underground Tunneling

Based on the City of Houston Monumentation Map, the proposed waterline along Harrisburg will cross a railroad track near Velasco. At this location the pipe line can be installed by using the technique of shield tunneling. Shield tunneling is most economical and widely used method of tunneling in soft ground conditions such as in Houston area. In this method, a shield (rigid steel cylinder) is forced ahead in steps, keeping pace with the progress of excavation and erection work, while at the same time providing a fully supported lining to the perimeter of the tunnel. A cycle of shield tunneling comprises the following items:

(i) excavation at the face and the provision of immediate temporary support to face as necessary;

(ii) advancing the complete shield in the direction of the excavation, developing the thrust from a previously erected lining; and,

(iii) placing another ring of permanent lining immediately behind the shield in the tail of the shield which spans from the main shield to the outside of the previous lining.

An estimate of the design pressure on tunnel liner for the crossings are presented on Plate 15. The vertical loads on the tunnel liner due to railroad traffic, as given below, should be added to the liner loads presented on Plate 15.

TUNNEL LOADS DUE TO SINGLE RAILROAD

Depth of Cover, ft	Vertical Pressure ksf	Length of Tunnel Affected by Load, ft.
6	0.67	13
7	0.59	14
8	0.53	15
9	0.48	16
10	0.44	17

The estimated length of the tunnel centered beneath the railroad track over which the load is applied is also shown in the above table. The loads and corresponding affected tunnel length were calculated assuming 70-ton capacity cars with a loaded weight of 212,000 lbs. The load was assumed to be distributed over an area of 8 ft (along Harrisburg) by 19.5 ft (along railroad track) at a depth of 1 ft and over an increasing area defined by 2 (V) : 1 (H) slopes with increasing depth. The loads shown assume only one railroad track at a given location. Where more than one track exists at a given location, the tunnel liner should be designed assuming the design loading on each track.

FAULTS IN THE VICINITY OF SITE

A fault is commonly defined as a break and displacement of various soil or rock layers of the earth due to subsurface movements. Within the Gulf Coast Plain of Texas and Louisiana, several faults are known to exist and these faults have resulted in broken ground surfaces, broken pavements, and damage to buildings and other structures.

The U. S. Geological Survey has mapped over 150 separate faults totaling more than 140 miles in length in the Houston area. It is estimated that at least 50 other surface faults have been recognized in the area; their locations have either not been published or have been published at a scale not suitable for general use. Many faults do not extend to the surface.

A fault may be inactive along all or part of its length, which means there is either no recent observable movement of the surface along the fault trace, or the fault does not extend to the surface. Any fault that has appeared at the surface or has broken or displaced man-made structures is considered to be active. The vertical movement of typical active faults average over a number of years ranges from about 0.25 in. to more than 1.0 in. per year. Horizontal movement is generally about one-fourth to one half the vertical movement. These surface movements generally occur in a band of significant width. The width of the movement zone varies with each fault and along a particular fault and it may increase with increasing displacement. When active, faults in the Houston area move intermittently by a "creep" process that preludes violent movements, such as earthquakes, resulting from faults that pass through hard rock.

Review of surface and subsurface faults, as are now known to exist, was made from the maps published by U.S. Geological Survey and the data available from the GEOTEST library. The primary objective of this study was to evaluate all available information from published reports, open file reports and information not generally available in published form. A review of aerial photographs was also made.

Based on the available information and our knowledge of the faults in the general vicinity, the nearest known surface fault, Pecore Fault is located about 3 miles northwest of Harrisburg at Dowling (See Plate 16). The nearest known subsurface fault is located about 2.2 miles northeast of the subject site. The subsurface fault is about 8000 to 8400 feet deep and should not affect any development at this site.

No surface fault was encountered at the site during the field investigation. A detailed fault investigation was beyond the scope of this study.

SUBSIDENCE

Based on the maps published by the Harris-Galveston Coastal Subsidence District, the amount of subsidence (to date) in the vicinity of this site has been about 5 feet. The contours of land surface subsidence from 1906 and 1978 for the Harris Galveston Coastal Subsidence District are presented on Plate 17.

MISCELLANEOUS

Variation in Soil Conditions

The subsurface conditions and the design information contained in this report are based on the test borings made at the time of drilling at specific locations. However, some variation in soil conditions may occur between test borings. The depth of the ground water level may be expected to vary with environmental variation such as frequency and magnitude of rainfall.

Construction Considerations

To a degree, the performance of the project is dependent upon the procedures and quality of construction. Also engineering analyses provided herein for design and construction are largely empirical. As most of the excavation for the construction of water mains will be performed on city streets and adjacent to the existing structures, it is therefore recommended that ground loss or movement should be monitored very closely during construction.

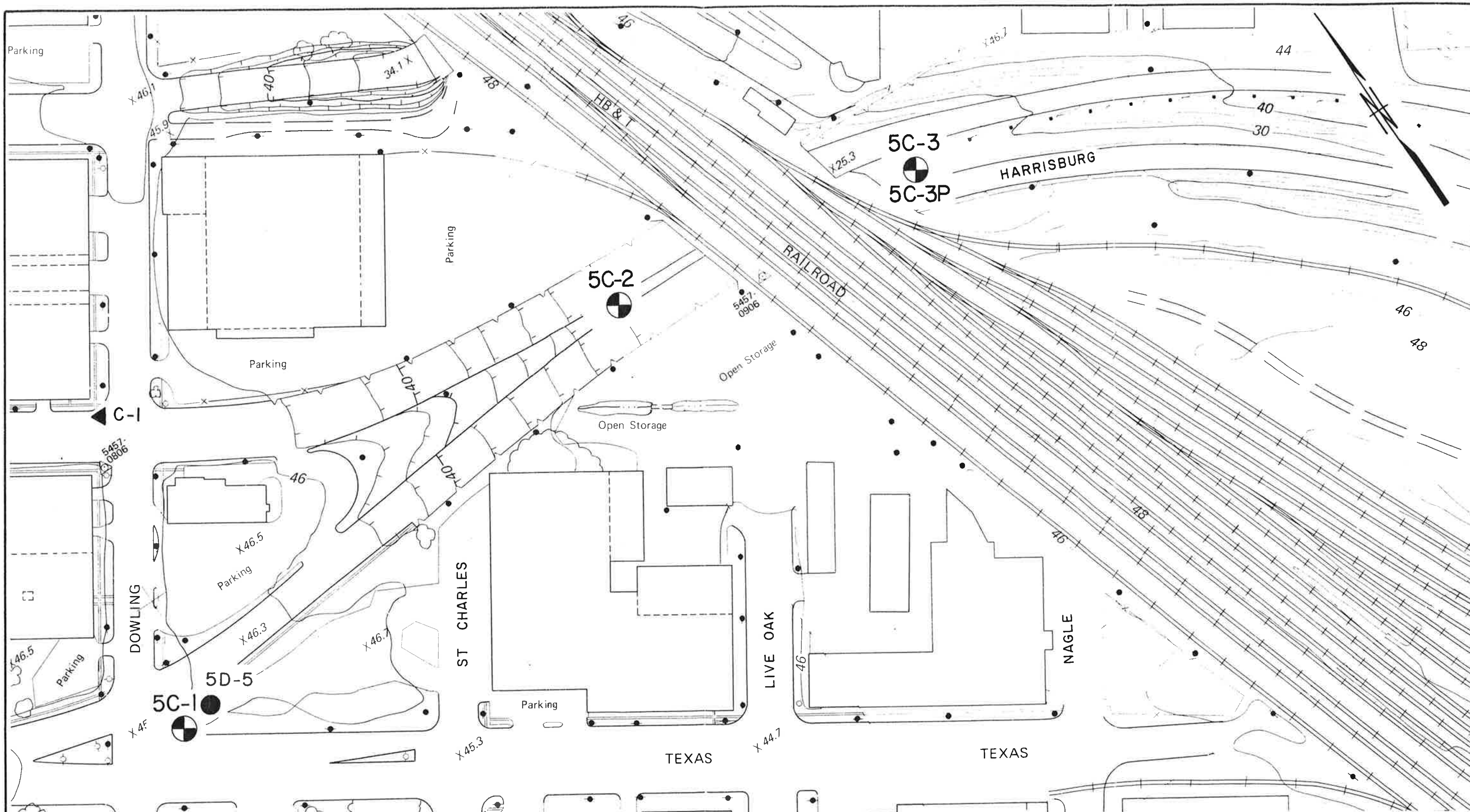
Both deformation of the bracing and heave of the excavation bottom are accompanied by subsidence of the soil adjacent to the excavation. This is known as lost ground. While in many instances a moderate subsidence is of no consequence, in others even a slight movement of the soil can result in damage to adjacent buildings. In some instances lost ground results from running of sands in a "quick" condition, in still other cases it may be caused by the slow , plastic creep of clays that are strong to stand in open excavations with no bracing at all.

Before any excavation that could cause damage to adjacent structures is begun, a survey should be made to determine the condition of those structures. The location, elevation, and size of all building cracks should be recorded and photographs secured. This information can do much to prevent annoying and expensive lawsuits that often arise during excavation work.

During construction, level readings should be made on points adjacent to the excavation to check the possibility of settlement that might go unnoticed otherwise. The bench mark should be located far enough from the excavation so that it will not subside and produce erratic level readings. A distance of at least five times the depth of the excavation from the excavation should be sufficient.

If settlement is caused by deformation, it can be reduced by tightening the bracing system or by prestressing it against the soil. If the bottom heaves, it can be prevented by driving the sheeting deeper and by loading the portions of the bottom of the excavation not actually involved in construction with the excavation waste or piles of sand. If running of sands occurs in a "quick" condition, it can be prevented by drainage to relieve the excessive hydrostatic pressure.

ILLUSTRATIONS



LEGEND ◀ Standard Electronic Friction Cone by Others, See Plates A-18 and A-19

● Boring by Others, See Plate A-20

⊕ 5C-3 : Soil Boring, and 5C-3P: Piezometer

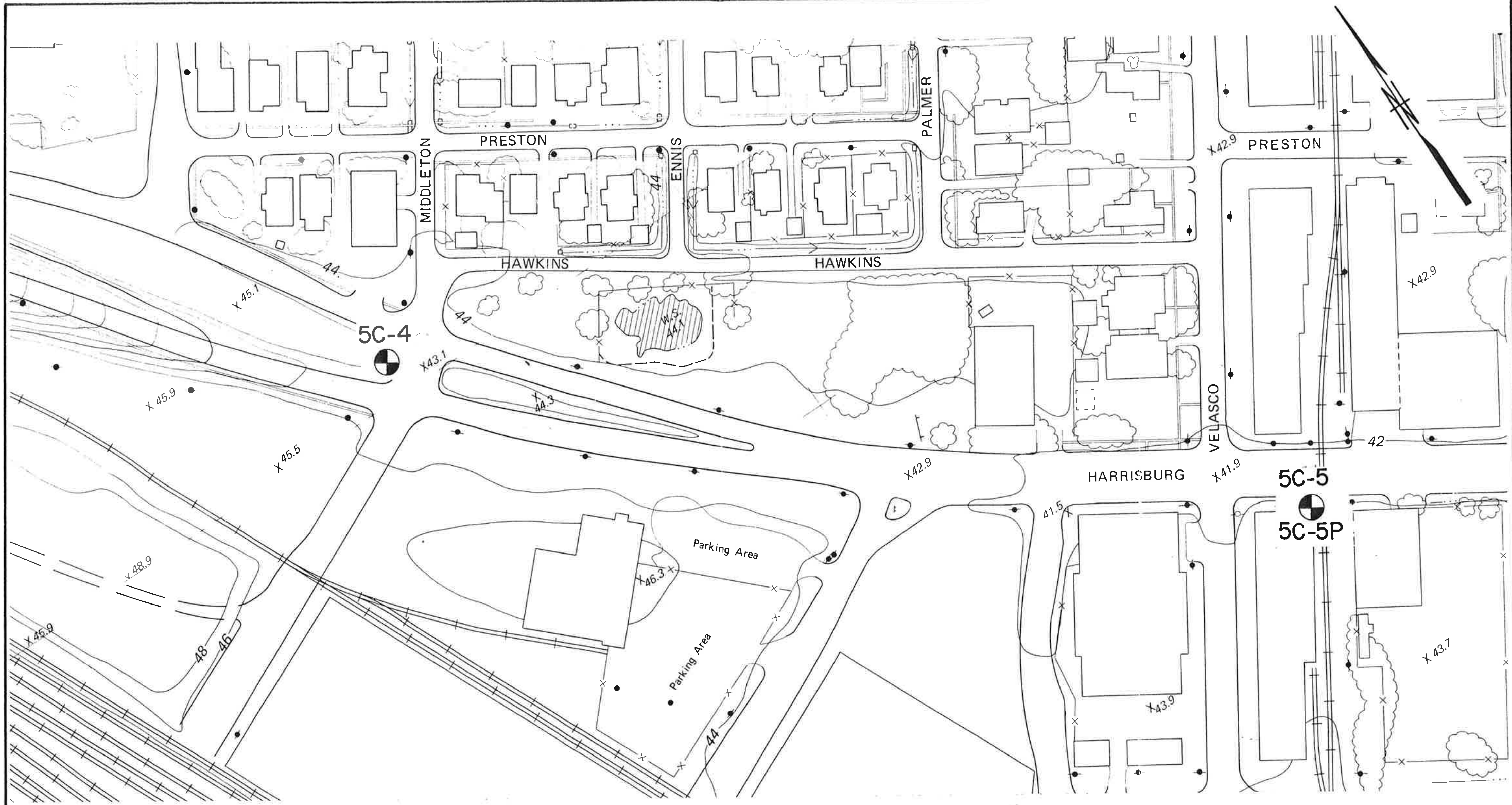
~30~ Contour Interval = 2' (Ref : C. O. H. Monumentation Map)

PLAN OF BORINGS

EAST WATER PROGRAM - CONTRACT 5C

SCALE: 1" = 100'

Geotest Engineering, Inc.



LEGEND

5C-5: Soil Boring, and 5C-5P: Piezometer

~42~ Contour Interval = 2' (Ref: C.O.H. Monumentation Map)

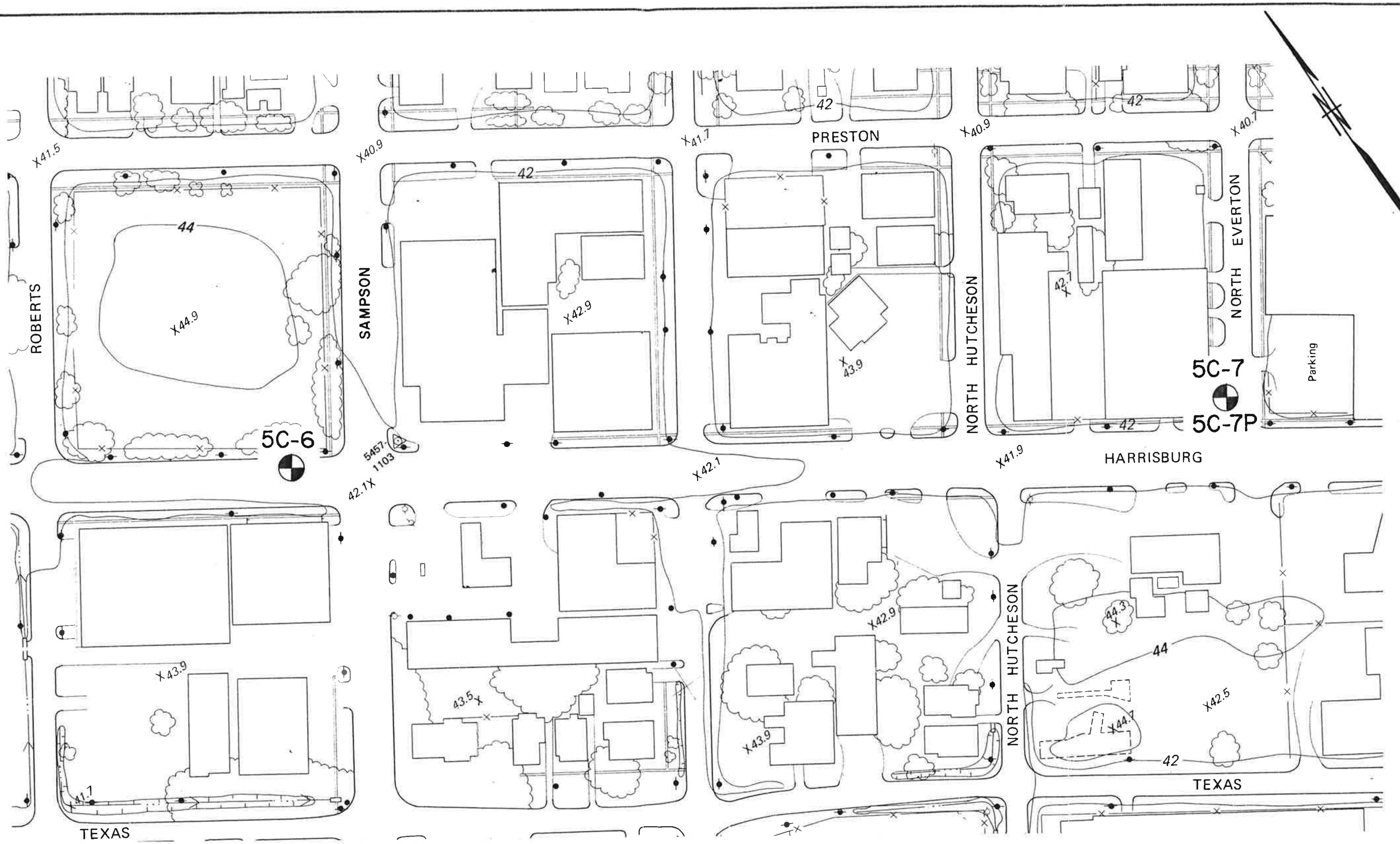
PLAN OF BORINGS

EAST WATER PROGRAM - CONTRACT 5C

SCALE: 1" = 100'

Geotest Engineering, Inc.

PLATE 1B



LEGEND

5C-7: Soil Boring, and 5C-7P: Piezometer

~42~ Contour Interval = 2' (Ref: C.O.H. Monumentation Map)

Geotest Engineering, Inc.

PLAN OF BORINGS

EAST WATER PROGRAM - CONTRACT 5C

SCALE: 1" = 100'

PLATE 1C

LEGEND

5C-7 : Soil Boring, and 5C-7P : Piezometer

~42~ Contour Interval = 2' (Ref : C.O.H. Monumentation Map)

PLAN OF BORINGS

EAST WATER PROGRAM - CONTRACT 5C

SCALE : 1" = 100'

Geotest Engineering, Inc.

PLATE 1D

LEGEND

- Boring by Others, See Plate A-21
- 5C-11: Soil Boring, and 5C-11P: Piezometer
- ~42~ Contour Interval = 2' (Ref: C. O. H. Monumentation Map)

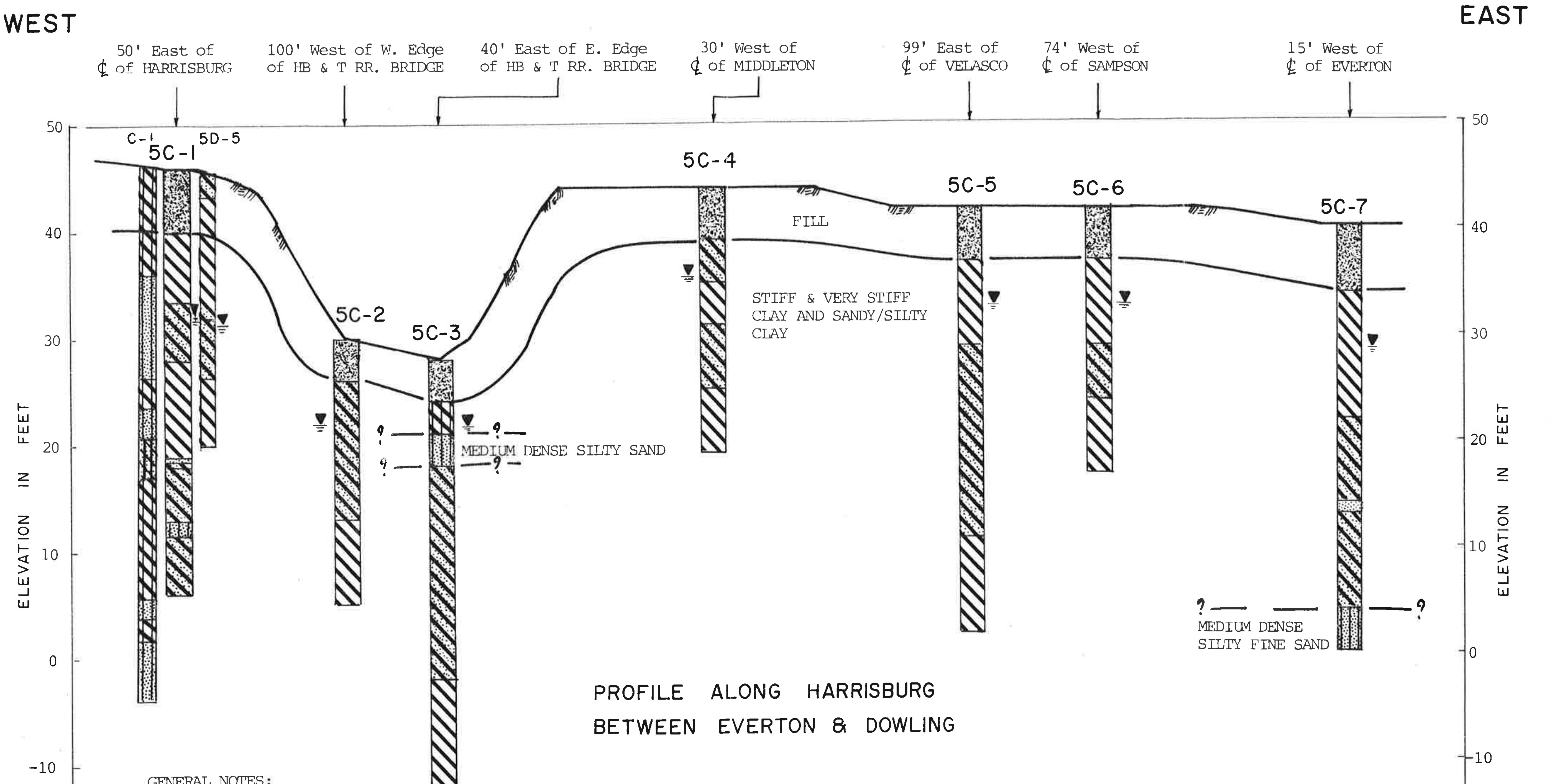
PLAN OF BORINGS

EAST WATER PROGRAM - CONTRACT 5C

SCALE: 1" = 100'

Geotest Engineering, Inc.

PLATE 1E



PROFILE ALONG HARRISBURG
BETWEEN EVERTON & DOWLING

GENERAL NOTES:

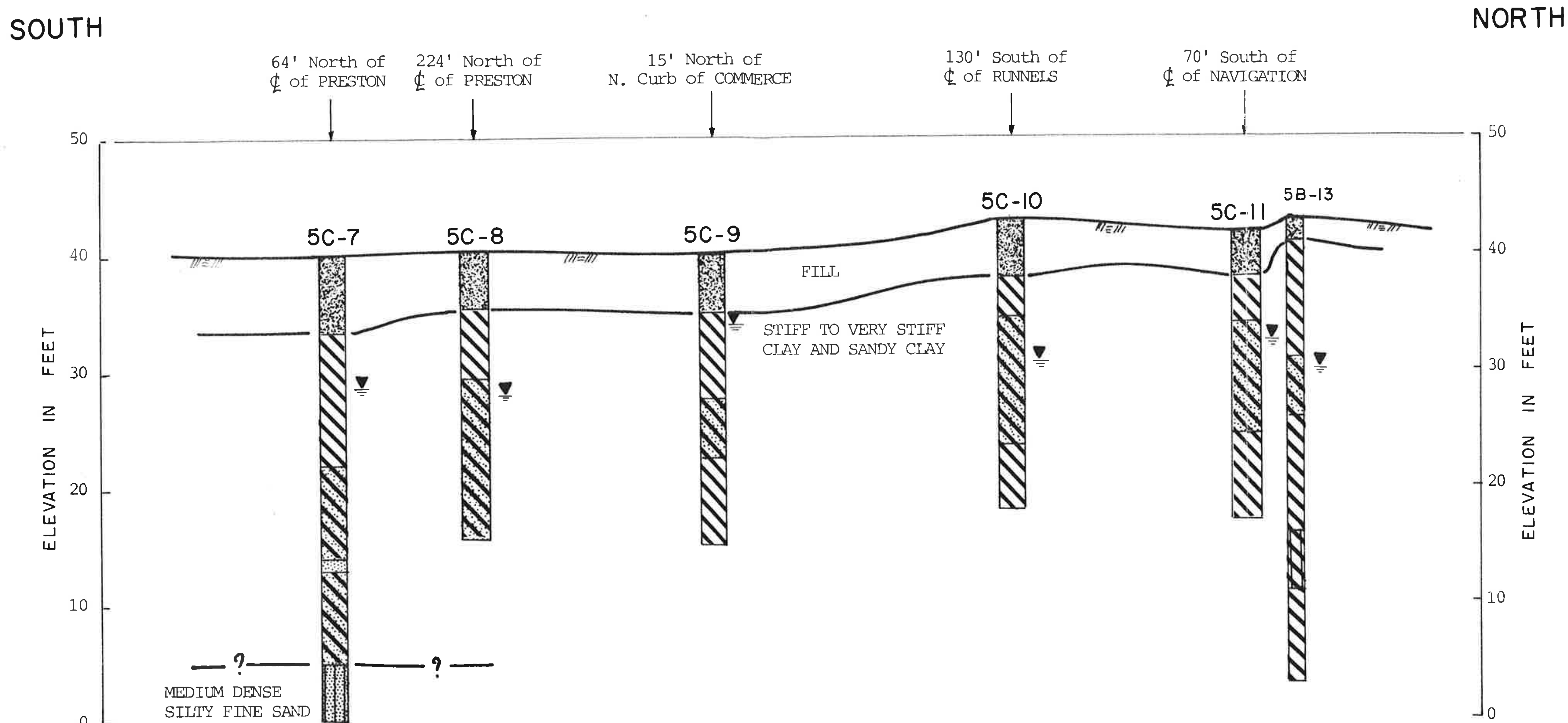
1. See Plates 1A, 1B & 1C for location of Borings
2. Ground water level shown by ▼
3. Data concerning subsurface conditions have been obtained at boring location only. Actual conditions at locations between borings may differ from inferred profile shown here.
4. See Plate A-12 for Legend

GENERALIZED SOIL PROFILE

EAST WATER PROGRAM - CONTRACT 5C

HORIZONTAL SCALE: 1" = 400'

Geotest Engineering, Inc.



PROFILE ALONG EVERTON
BETWEEN NAVIGATION & HARRISBURG

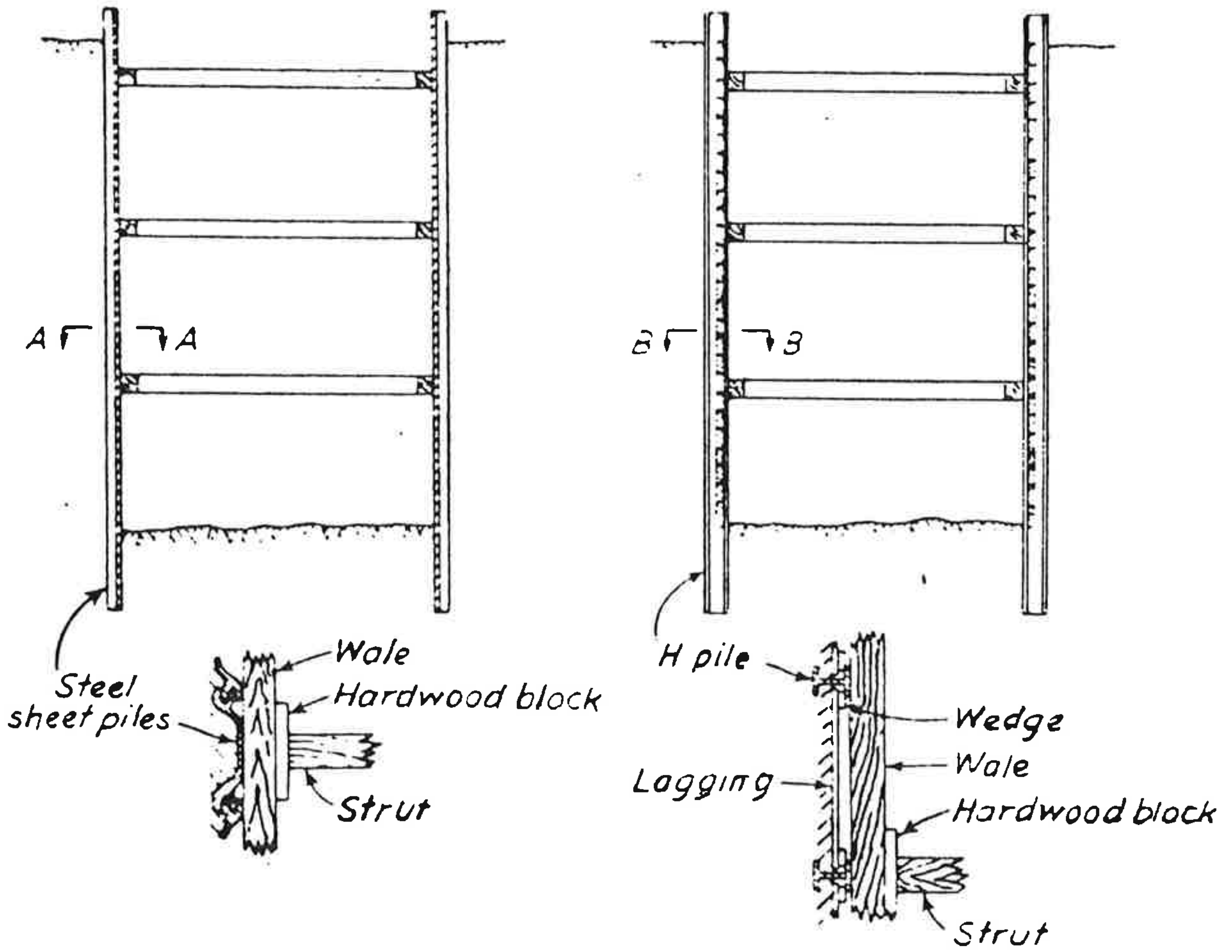
GENERAL NOTES

1. See Plates 1D & 1E for location of Borings
2. Ground water level shown by ▼
3. Data concerning subsurface conditions have been obtained at boring location only. Actual conditions at locations between borings may differ from inferred profile shown here.
4. See Plate A-12 for Legend

GENERALIZED SOIL PROFILE

EAST WATER PROGRAM - CONTRACT 5C

HORIZONTAL SCALE : 1" = 400'

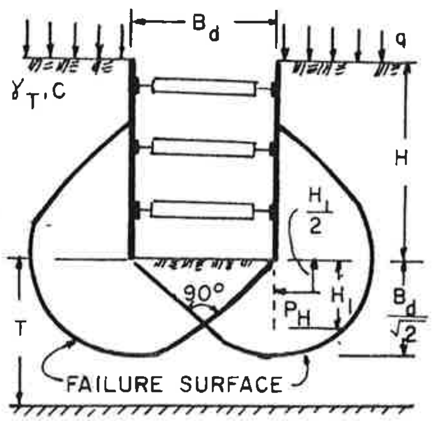


Section A-A

Section B-B

TYPICAL BRACING SYSTEMS FOR DEEP EXCAVATIONS

CUT IN CLAY, DEPTH OF CLAY UNLIMITED ($T > 0.7B_d$)



If sheeting terminates at base of cut:

Safety factor, $F_s = \dfrac{N_c C}{\gamma_T H + q}$

N_c = Bearing capacity factor, which depends on dimensions of the excavation : B_d, L and H (use $N_c = 6.5$ for contract 5C)

C = Undrained shear strength of clay in failure zone beneath and surrounding base of cut

q = Surface surcharge

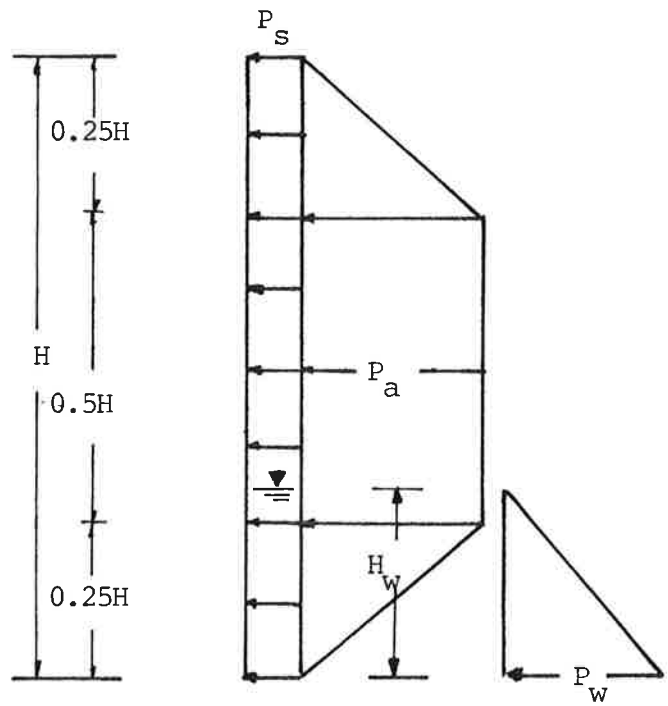
If safety factor is less than 1.5, sheeting must be carried below base of cut to insure stability

Force on buried length:

If $H_1 > \dfrac{2}{3}\,\dfrac{B_d}{\sqrt{2}}$, $PH = 0.7\,(\gamma_T HB_d - 1.4CH - \pi CB_d)$

If $H_1 < \dfrac{2}{3}\,\dfrac{B_d}{\sqrt{2}}$, $PH = 1.5H_1\,(\gamma_T H - \dfrac{1.4CH}{B_d} - \pi C)$

STABILITY OF BOTTOM FOR BRACED CUT



$P = P_a + P_s + P_w$

Where:

P = Lateral Pressure, psf;

P_a = Active Earth Pressure, psf, $P_a = 0.3\gamma H$;

P_s = Pressure due to Surcharge, psf, $P_s = 0.3q_s$;

P_w = Hydrostatic Pressure, $P_w = \gamma_w H_w$;

H = Depth of Braced Excavation, feet;

H_w = Hydrostatic Head above Excavation Level, feet;

γ = Wet unit weight of soil, pcf (recommended value 125 pcf);

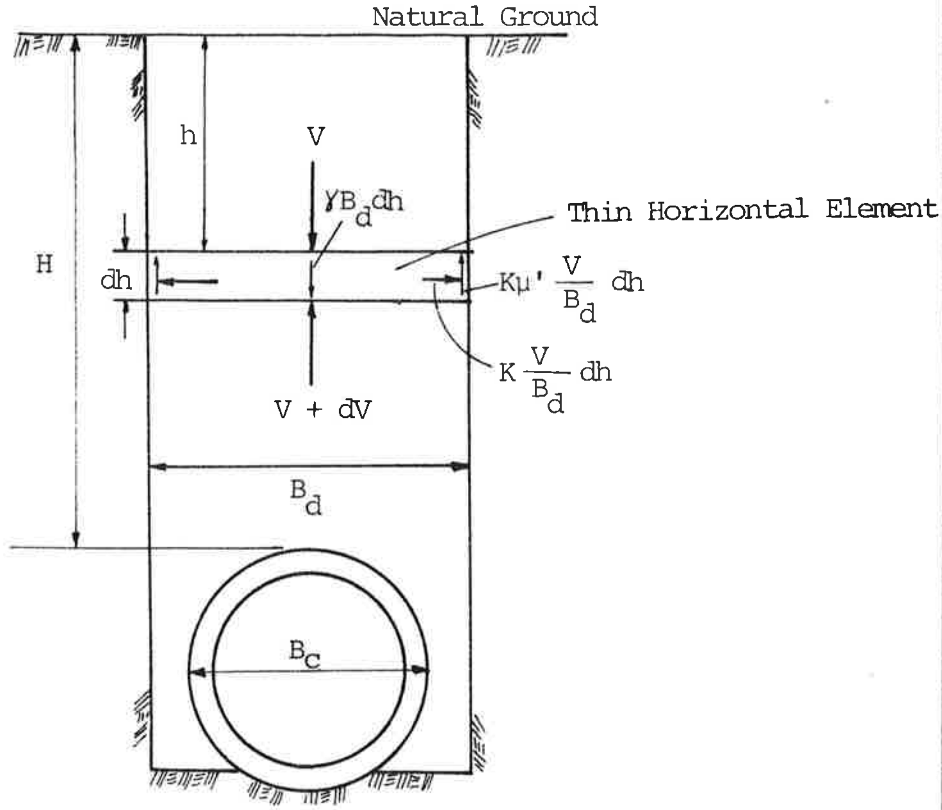
γ_w = Unit weight of water, pcf = 62.5 pcf;

q_s = Surcharge Load, psf, q_s = 500 psf

STREET	H (ft.)		H_w (ft.)		P_a (psf)		P_w (psf)		P_s
	max.	min.	max.	min.	max.	min.	max.	min.	(psf)
HARRISBURG	19	15	13	9	715	565	815	565	150
EVERTON	19	15	13	9	715	565	815	565	150

EARTH PRESSURE DIAGRAM FOR BRACED CUT

Geotest Engineering, Inc.



Where: γ = wet unit weight of backfill, pcf

V = vertical pressure on any horizontal plane in backfill, lb/linear ft.

B_c = outside diameter of conduit, feet

B_d = width of ditch at top of conduit, feet

H = height of fill above top of conduit, feet

h = distance from ground surface down to any horizontal plane in backfill, feet

$\mu' = \tan \phi'$ = coefficient of friction between backfill and sides of ditch

K = ratio of active lateral unit pressure to vertical unit pressure

dh = height of a thin horizontal element of fill material located at any depth h below the ground surface, feet

$V + dV$ = vertical pressure on the bottom of the element, lb/linear ft.

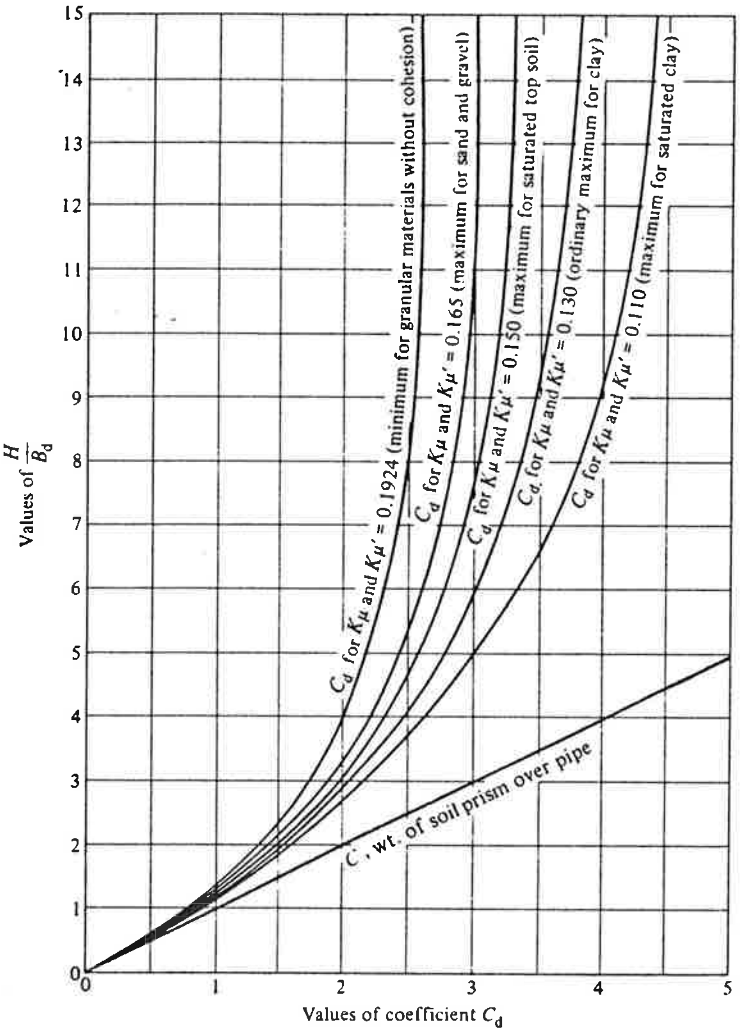
$\gamma B_d dh$ = weight of element, lb/linear ft.

$K \frac{V}{B_d} dh$ = lateral pressure on each side of the element, lb/linear ft.

$K\mu' \frac{V}{B_d} dh$ = upward shearing forces, lb/linear ft.

FREE BODY DIAGRAM FOR DITCH CONDUIT

Job No.



Where: H = depth of cover above top of conduit

B_d = width of trench at top of conduit

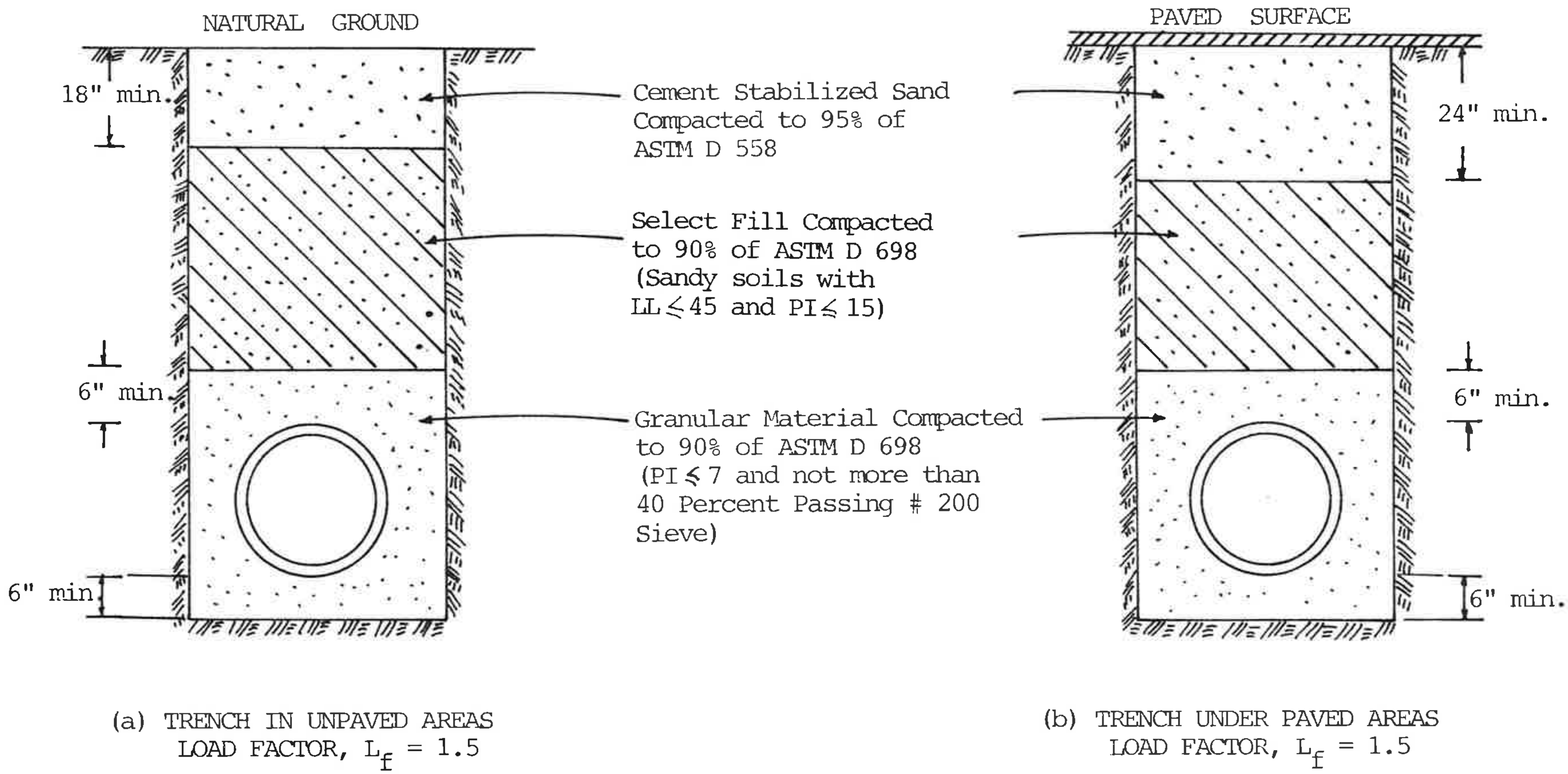
K = active earth pressure coefficient = $\tan^2(45° - \frac{\emptyset}{2})$

$\emptyset$ = internal friction angle of soil

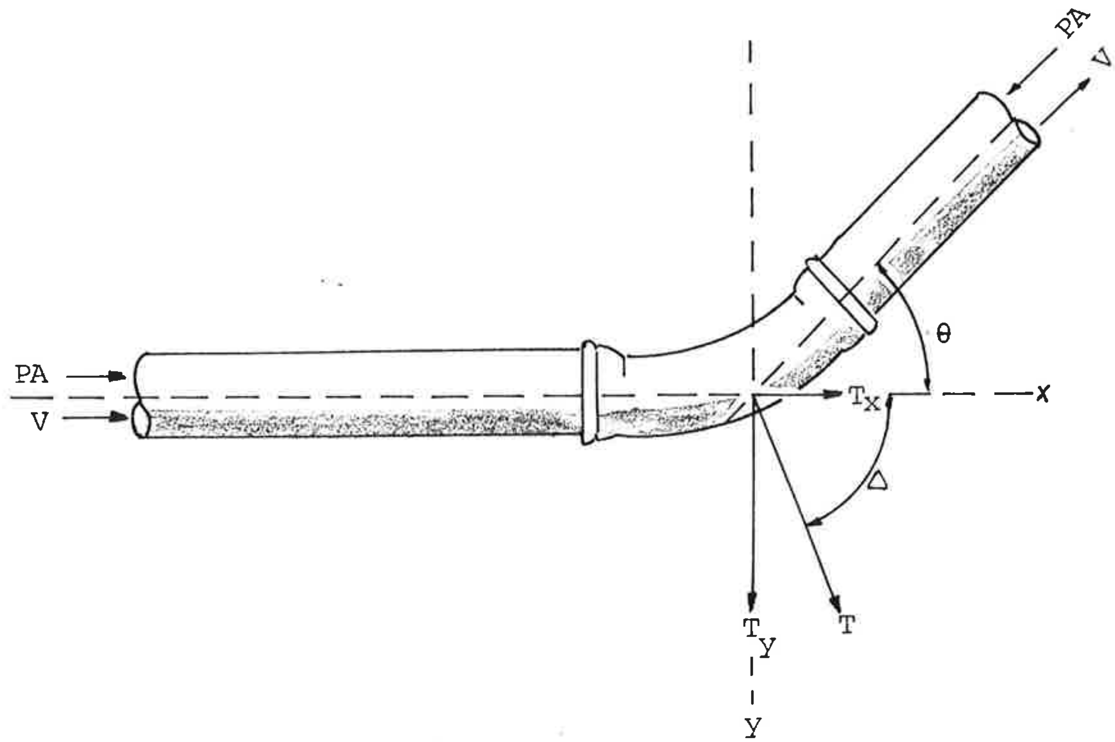
μ = $\tan\emptyset$ = coefficient of internal friction of fill material

μ' = $\tan\emptyset'$ = coefficient of friction of fill material and sides of ditch

LOAD COEFFICIENT FOR DITCH CONDUITS



RECOMMENDATIONS
FOR
BEDDING, ENCASEMENT AND BACKFILLING



T_x = PA (1 - COS θ)

T_y = PA SIN θ

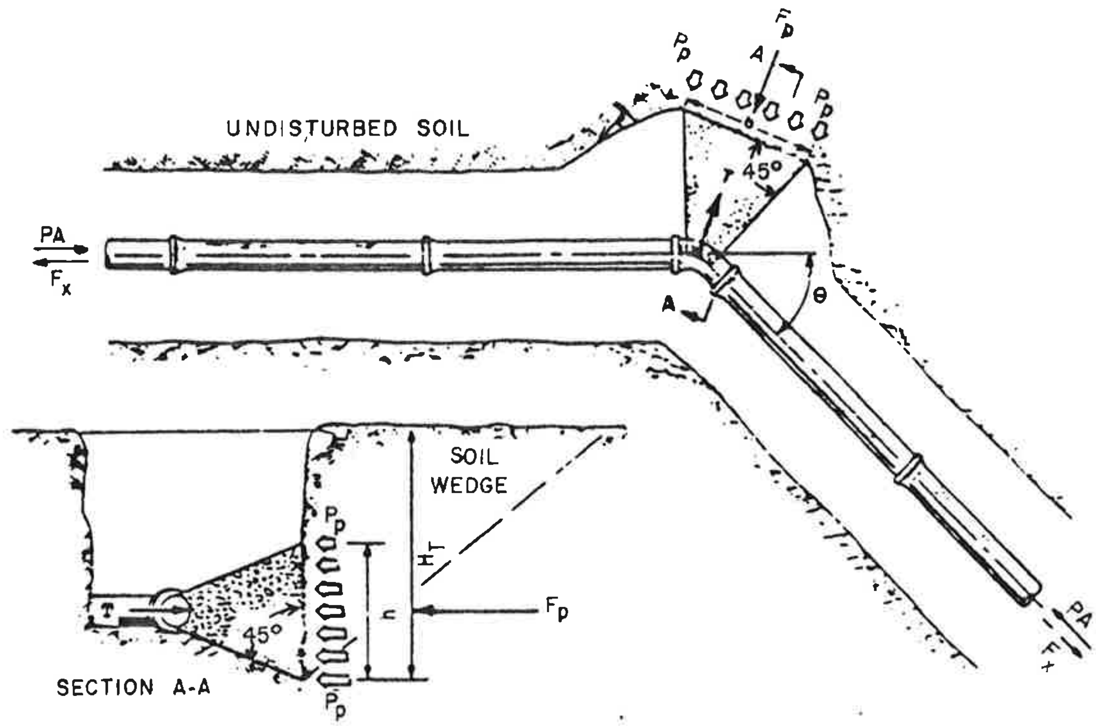
T = 2 PA SIN $\frac{\theta}{2}$

Δ = (90 - $\frac{\theta}{2}$)

Where:

- T is the resultant force on the bend
- T_x is the component of thrust force in x-direction
- T_y is the component of thrust force in y-direction
- P is the maximum sustained pressure
- A is the pipe cross-sectional area
- θ is the bend deflection angle
- Δ is the angle between T and X-axis
- V is the fluid velocity

THRUST FORCES ACTING ON A BEND



Where:

T	is the resultant thrust force on the bend
F_b	is the resistance force developed by passive soil pressure
A_b	is the minimum bearing area of block base
h	is the height of the thrust block
b	is the width of thrust block
A	is pipe cross-sectional area
θ	is bend deflection angle
P_p	is the passive soil pressure
H_T	is the depth to the bottom of block
γ	is the soil unit weight
K_p	is the passive earth pressure coefficient
Ø	is the soil internal friction angle
C	is the soil cohesion
S_f	is a factor of safety (usually 1.5)
H_c	is the mean depth from ground surface to the plane of resistance (center of bearing area of a thrust block)
P	is maximum sustained pressure

Required Bearing Area

$$A_b = hb = \frac{S_f 2PA \sin\frac{\theta}{2}}{P_p}$$

Required Block Width

$$b = \frac{2S_f PA \sin\frac{\theta}{2}}{h\,P_p}$$

Where

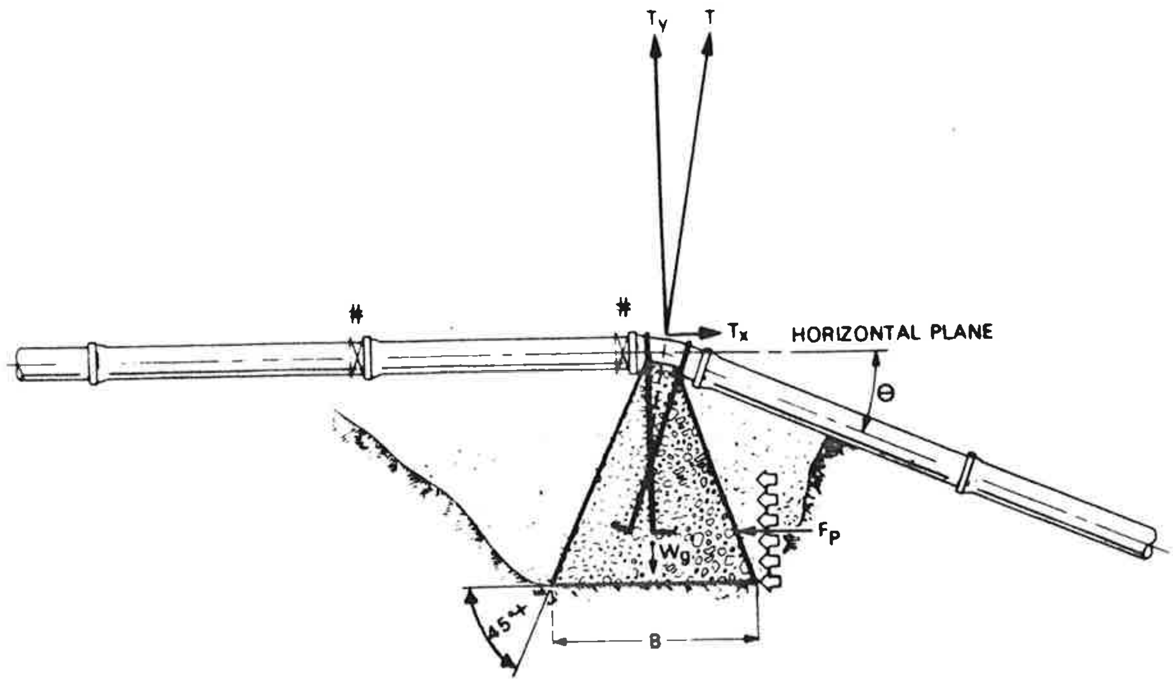
$$P_p = \gamma H_c K_p + 2C\sqrt{K_p}$$

$$K_p = \mathrm{Tan}^2\,(45° + Ø/2)$$

For $h = 1/2\ H_T$

$$b = \frac{S_f 2PA \sin\frac{\theta}{2}}{3/8\ \gamma H_T^2\ K_p + CH_T\sqrt{K_p}}$$

DESIGN PARAMETERS FOR BEARING THRUST BLOCK



Restrained joints may be used when $T_x > F_p$

Required volume of gravity block

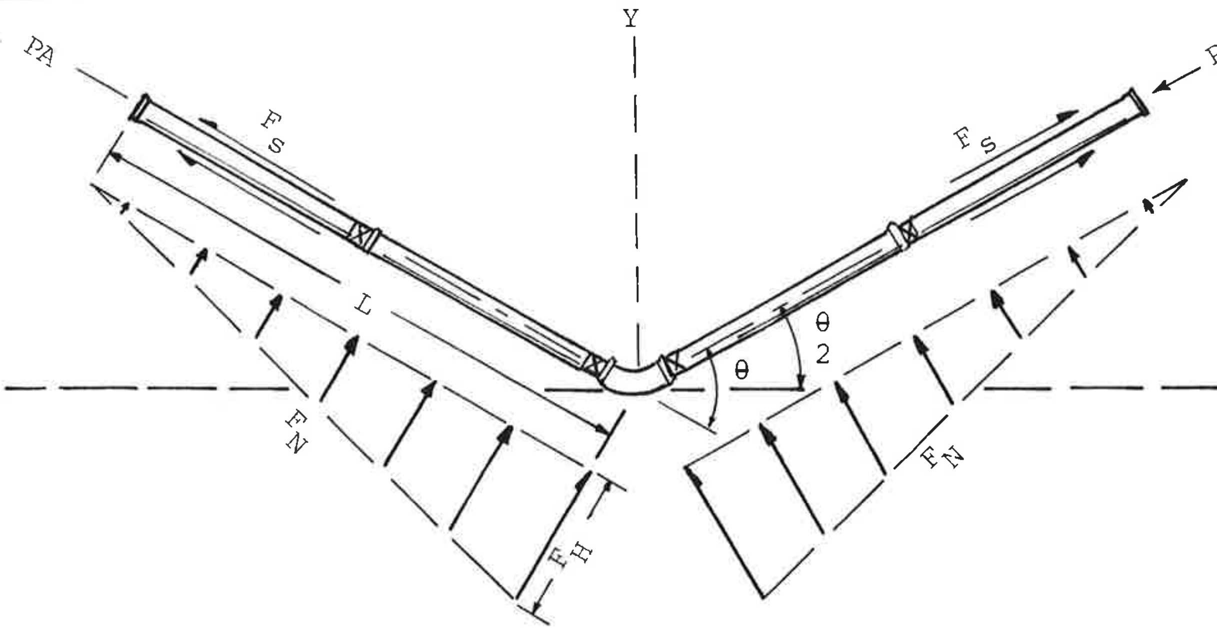
$$V_G = \frac{S_f PA \sin\theta}{W_m}$$

Where

- T = Resultant thrust force
- T_x = X thrust force component
- T_Y = Y thrust force component
- V_G = Volume of gravity block
- P = Maximum sustained pressure
- A = Pipe cross-sectional area
- θ = Bend deflection angle
- W_m = Density of block material
- W_g = Weight of gravity block
- F_p = Resisting force developed by passive soil pressure
- B = Gravity block base dimension
- S_f = Safety factor (usually 1.5)

DESIGN PARAMETERS
FOR
GRAVITY THRUST BLOCK

Job No.



Required Length

$$L = \frac{S_f \; KPA}{KF_s + B_c p_p}$$

Where

$$K = 4 \tan \frac{\theta}{2}$$

$$F_s = A_p f + W \tan\delta$$

and

$$p_p = \gamma H_c K_p + 2C\sqrt{K_p}$$

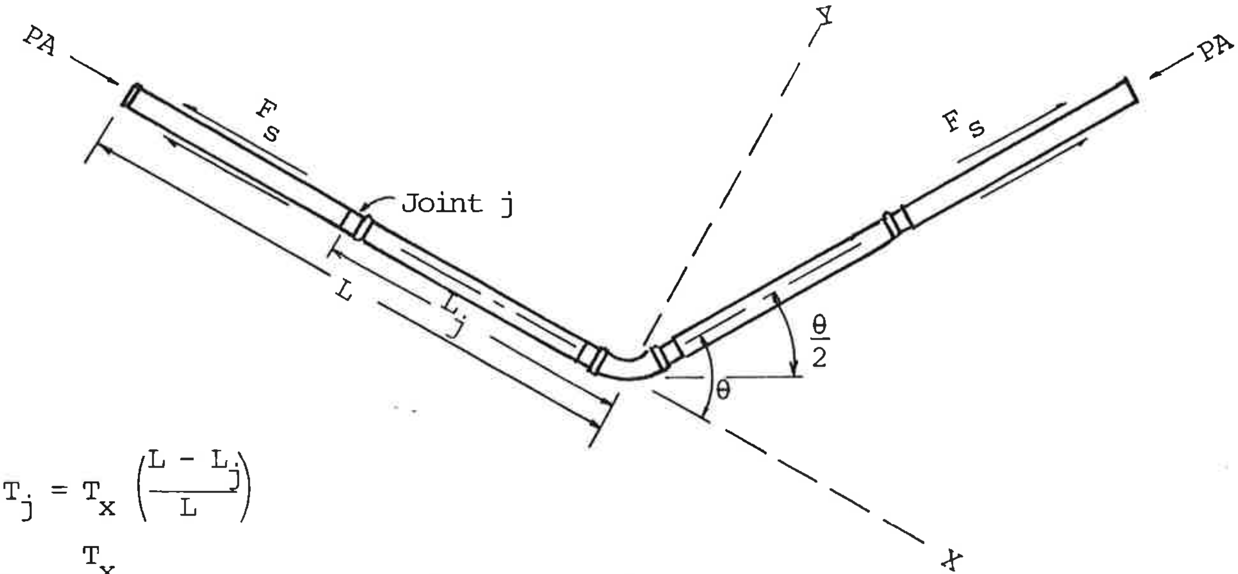
$$K_p = \tan^2 (45° + \emptyset/2)$$

$$A_p = \frac{\pi D}{2}$$

$$W = \gamma \pi H B_c R$$

Where
- L is the restrained pipe length on each side of the bend
- S_f is a factor of safety (usually 1.25)
- K is the bend coefficient
- F_s is the conduit frictional resistance per unit length
- p is the maximum sustained pressure
- A is pipe cross-sectional area
- θ is bend deflectional angle
- B_c is outside diameter of the conduit
- A_p is the conduit surface area per unit length (assume ½ the pipe circumference bears against the backfill soil)
- f is the cohesion between conduit and backfill
- W is the normal force on the pipe per unit length
- δ is the frictional angle between the conduit and the backfill soil
- p_p is the passive soil pressure
- γ is the soil unit weight
- H is the depth to top of conduit
- K_p is the passive earth pressure coefficient
- $\emptyset$ is the soil internal friction angle
- C is the soil cohesion
- H_c is the mean depth from ground surface to the plane of resistance (center line of a pipe)
- R is reduction factor depending on trench condition (generally 2/3)
- D is inside diameter of the conduit

DESIGN PARAMETERS FOR RESTRAINED JOINT



$$T_j = T_x \left(\frac{L - L_j}{L}\right)$$

and

$$L = \frac{T_x}{F_s}$$

Therefore

$$T_j = F_s\ (L - L_j)$$

and

$$F_s = A_p\ f + W \tan\delta$$

$$A_p = \frac{\pi D}{2}$$

$$W = \gamma \pi H B_c R$$

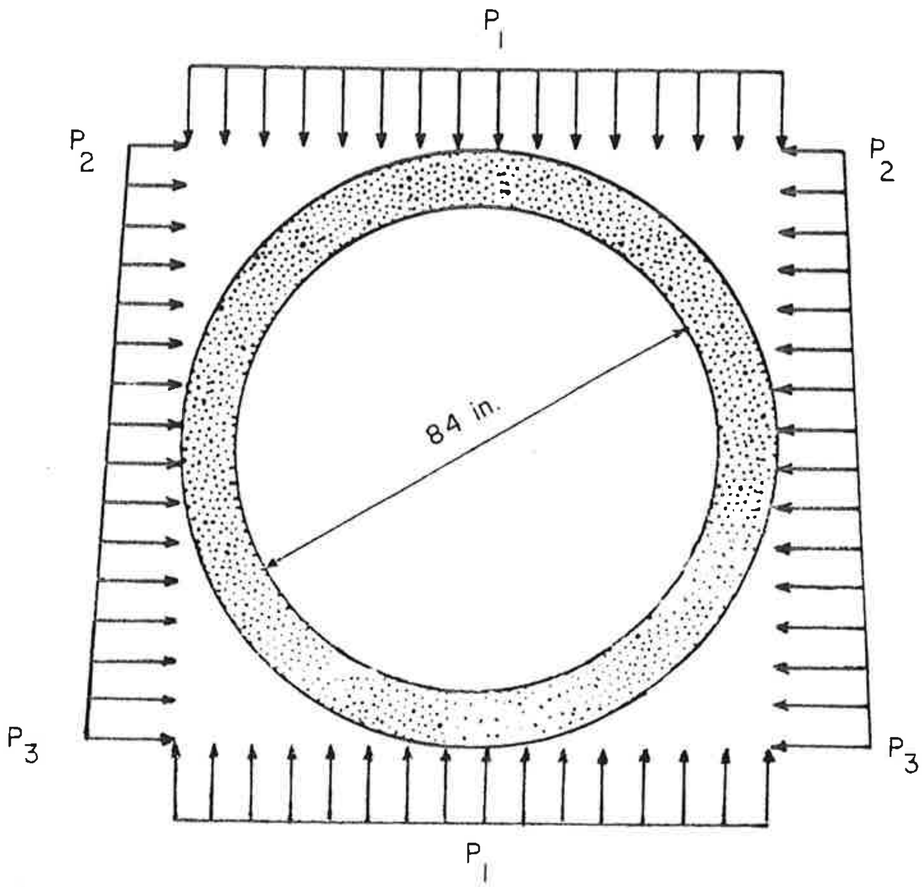
Where T_j = thrust force at joint
T_x = X thrust force component
F_s = unit conduit frictional resistance
L = restrained pipe length on each side of the bend
L_j = distance from bend to joint
P = maximum sustained pressure
A = pipe cross-sectional area
D = I.D. of pipe; B_c = O.D. of pipe
γ = soil unit weight
A_p = conduit surface area per unit length (assume ½ the pipe circumference bears against the backfill soil)
f = the cohesion between conduit and backfill soil, 0.5C
C = the soil cohesion
W = the normal force on the pipe per unit length
δ = the friction angle between the conduit and backfill soil, 0.75Ø
Ø = soil internal friction angle
R = reduction factor depends on trench condition (generally 2/3)

The required number of rods (N) can be determined by the following equation:

$$N = \frac{S_f T_j}{F}$$

Where N = the required number of rods
S_f = safety factor, 1.5
F = force developed per rod = SA_r
S = tensile stress of rod
A_r = cross-sectional area of rod

DESIGN PARAMETERS FOR TIE RODS



TUNNEL LOCATION		LINER LOAD (ksf)		
		P_1	P_2	P_3
Harrisburg at RR. tarck between Velasco and Roberts	max. min.	2.27 1.77	0.91 0.63	1.68 1.31

sp'

NOTE:

Assume 10-ft. cover for maximum liner load and 6-ft. cover for minimum liner load

TUNNEL LINER LOADS

Job No.: 14G152

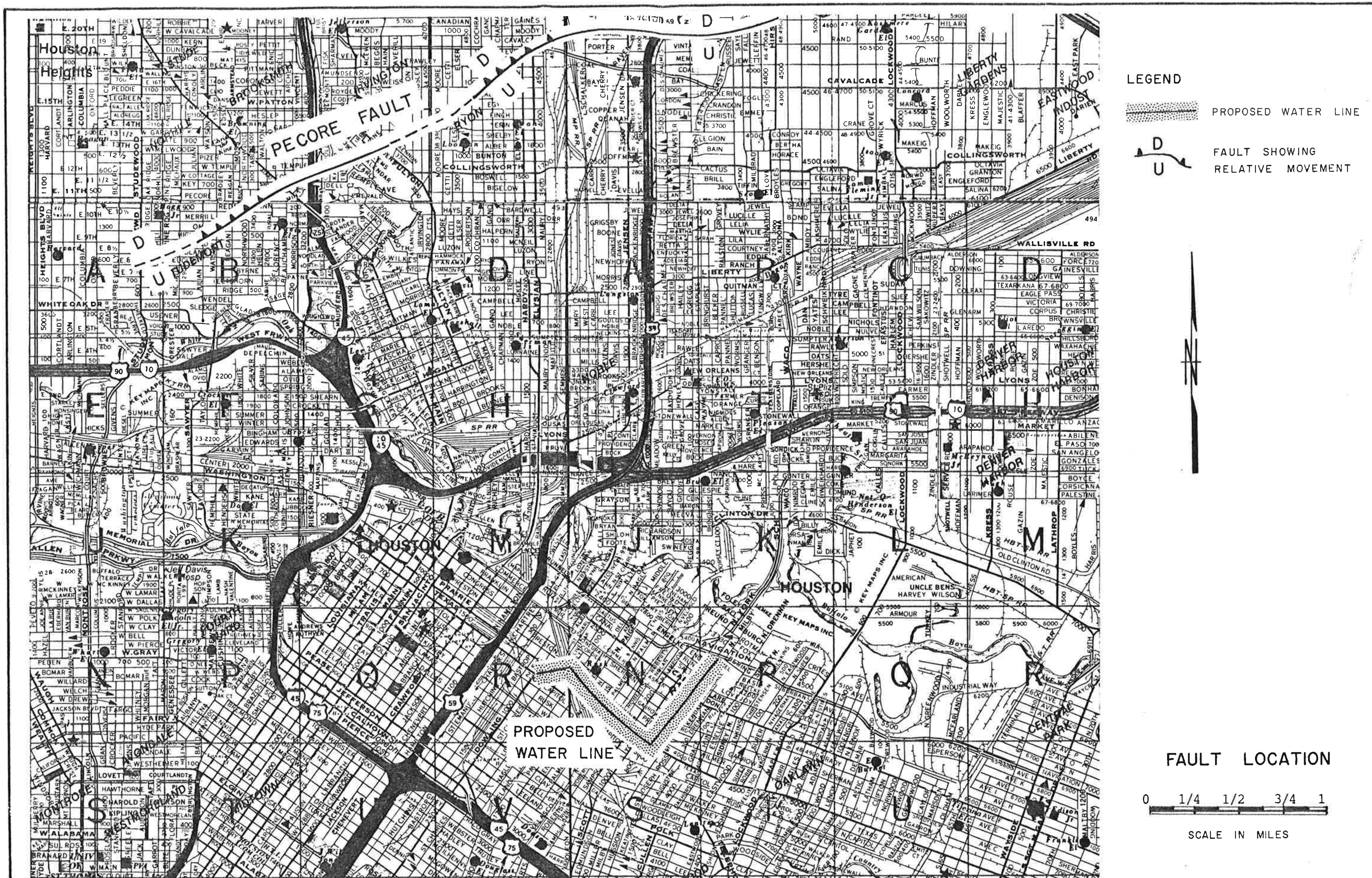
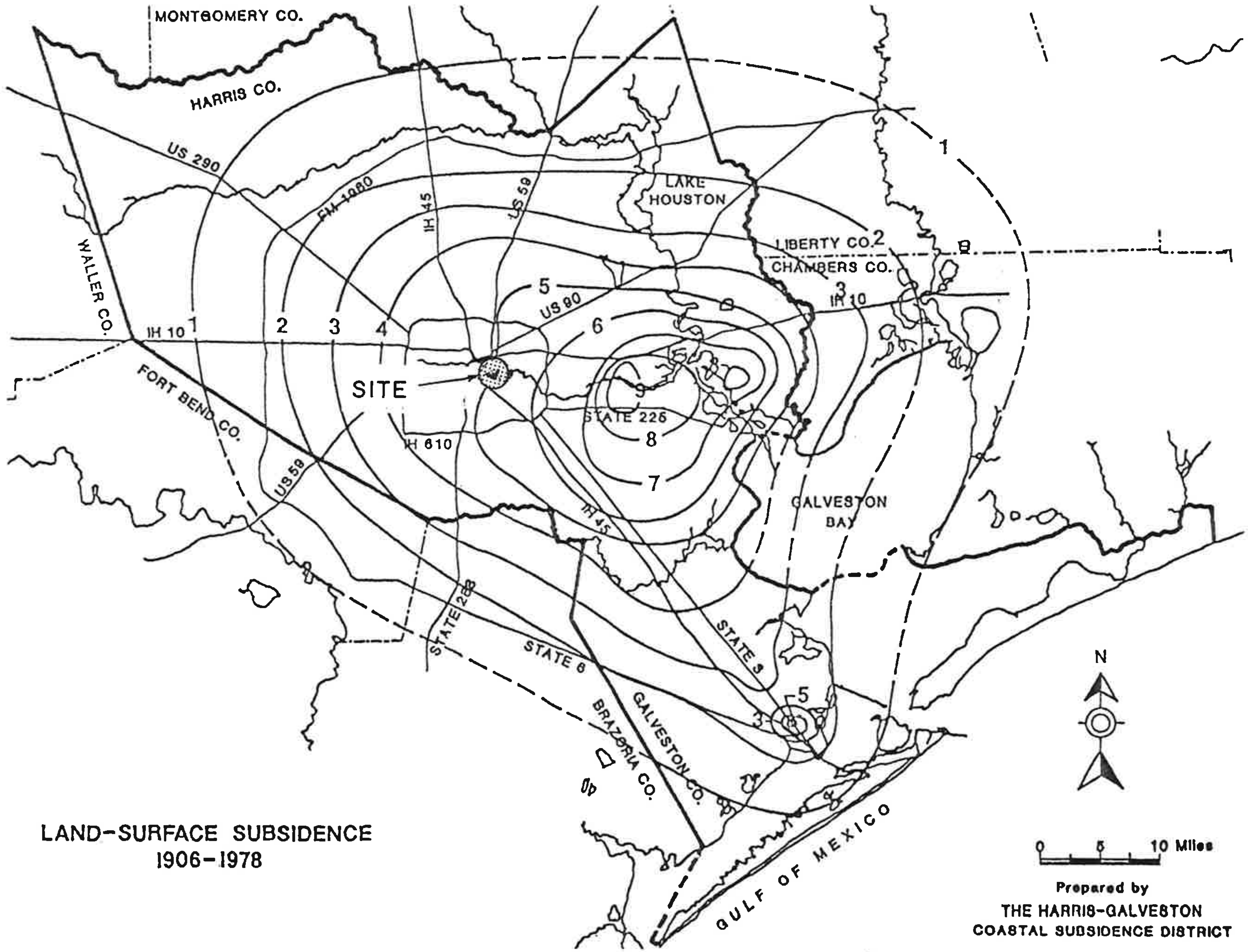


PLATE 16



LAND-SURFACE SUBSIDENCE
1906-1978

APPENDIX A

LOG OF BORING NO: 5C-1
(Drilled in Esplanade)

LOCATION: 50'E. of ℄ of Dowling
40'S. of ℄ of Harrisburg (E.Bound)

TYPE: 3-in.Shelby Tube; 2-in.Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 46'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0–5	Fill: stiff gray & tan clay w/ shell		9			
			19			
			23		69	43
5–13	Very stiff tan & light gray clay -w/calcareous nodules		29			
			25	100		
13–18	Very stiff & tan light gray sandy clay		15		40	22
18–28	Very stiff red & light gray clay -w/calcareous nodules 18'-25'		28		67	41
	-w/silt pockets 23'-28'		16			
	-sand layer 27'-27.5'					
28–40	Very stiff to stiff tan & light gray sandy clay		23	110		
	-tan silty fine sand layer 33'-34.5'		20			
	24% Passing #200 Sieve					
	-red & light gray w/calcareous nodules below 38'		18		41	23

SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
△ TORVANE
■ UU TRIAXIAL
0.5 1.0 1.5

COMPLETION DEPTH: 40'
DATE: 9-5-86

DEPTH TO WATER IN BORING: 14.5'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-2
(Drilled in Esplanade)

TYPE: 3-in. Shelby Tube; 2-in. Split Barrel.

LOCATION: 100'W. of W. Edge of HB&T R.R. Bridge along ₵ of Esplanade of Harrisburg

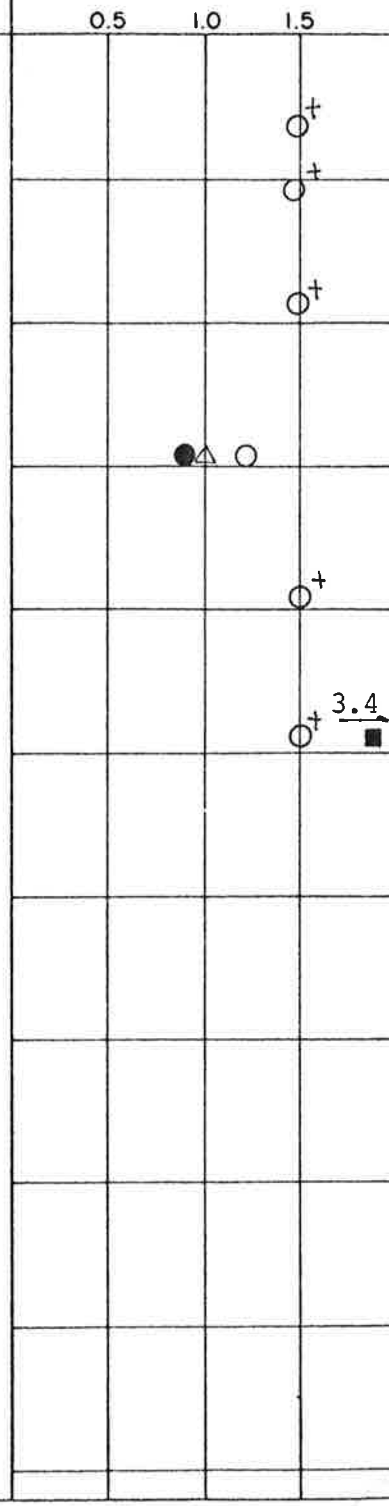
PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 30'

DEPTH, FEET	SYMBOLS	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0			Fill: very stiff brown sandy clay w/calcareous nodules & ferrous nodules		14		48	27
					14		51	30
5			Very stiff brown sandy clay -tan @ 6' -light gray & tan @ 7.5'		16			
10			-red & light gray clay, slickensided, 8.5'-12'		18		49	29
15			-tan & light gray below 13'		16	115		
20			Very stiff red & light gray clay, slickensided, w/calcareous nodules		21			
25					19	106		

SHEAR STRENGTH, TSF

○ HAND PENETROMETER
● UNCONFINED COMPRESSION
△ TORVANE
■ UU TRIAXIAL



COMPLETION DEPTH: 25'
DATE: 9-3-86

DEPTH TO WATER IN BORING: 8'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-3
(Drilled in Esplanade)

TYPE: 3-in.Shelby Tube; 2-in.Split Barrel.

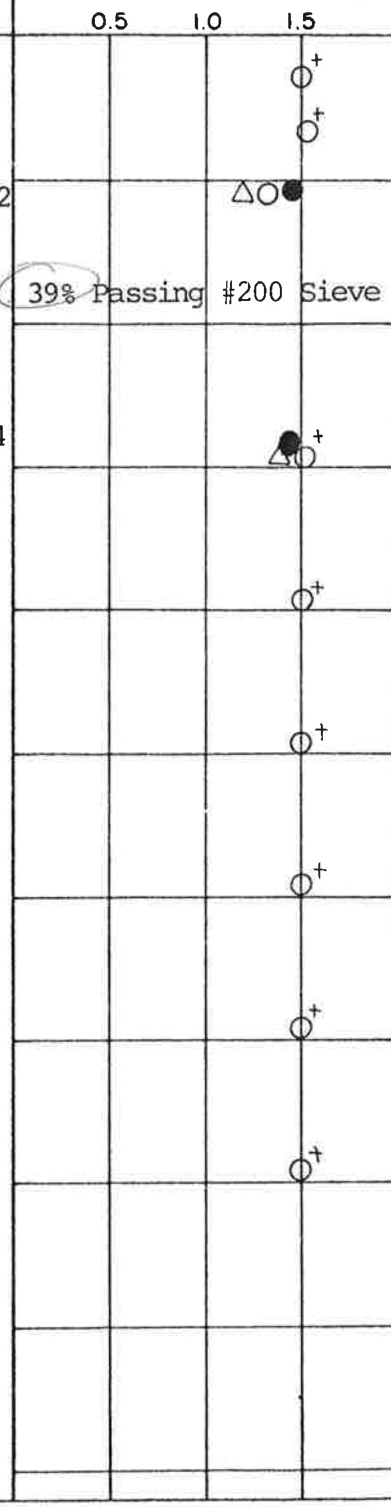
LOCATION: 40'E.of E.Edge of HB&T R.R.Bridge
30'N.of ₵ of Harrisburg (E.Bound)

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 28'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0–4	Fill: very stiff gray & tan sandy clay -tan & light gray below 2'		11		40	22
			15		36	19
4–7	Very stiff light gray silty clay		19	112		
7–11	Medium dense light gray silty fine sand w/silty clay seams		24			
	39% Passing #200 Sieve	21	22			
11–30	Very stiff tan & light gray sandy clay		16	114	44	25
			15			
	-red & light gray w/clay seams below 22'		17		44	25
			19			
30–40	Very stiff red & light gray clay, slickensided		24			
			24			

SHEAR STRENGTH, TSF



COMPLETION DEPTH: 40'
DATE: 9-4-86

DEPTH TO WATER IN BORING: 6.2'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-4
(Drilled in Esplanade)

LOCATION: 30'W.of ℄ of Middleton
45'N of ℄ of Harrisburg (E.Bound)

TYPE: 3-in.Shelby Tube; 2-in.Split Barrel.

PROJECT : East Water Program, Contract 5C

SURFACE ELEV. 44'

SHEAR STRENGTH, TSF: ○ HAND PENETROMETER; ● UNCONFINED COMPRESSION; △ TORVANE; ■ UU TRIAXIAL (scale 0.5, 1.0, 1.5)

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	SHEAR STRENGTH, TSF	LIQUID LIMIT	PLASTICITY INDEX
0–5	Fill: very stiff brown sandy clay w/calcareous nodules		11			36	19
			14		3.1		
5–9	Very stiff brown sandy clay -tan @ 7' -light gray & tan @ 8'		15	116			
			18				
			18	109	4.7		
9–13	Very stiff red & light gray clay, slickensided, w/calcareous nodules						
13–19.5	Very stiff tan & light gray sandy clay		16			35	18
19.5–25	Very stiff red & light gray clay, slickensided, w/calcareous nodules		16				
			18	108			

COMPLETION DEPTH: 25'
DATE: 9-3-86

DEPTH TO WATER IN BORING: 8'

Geotest Engineering Inc.

PLATE A-4

LOG OF BORING NO: 5C-5
(Drilled in Shoulder)

LOCATION: 99'E. of ℄ of Velasco
45'S. of ℄ of Harrisburg

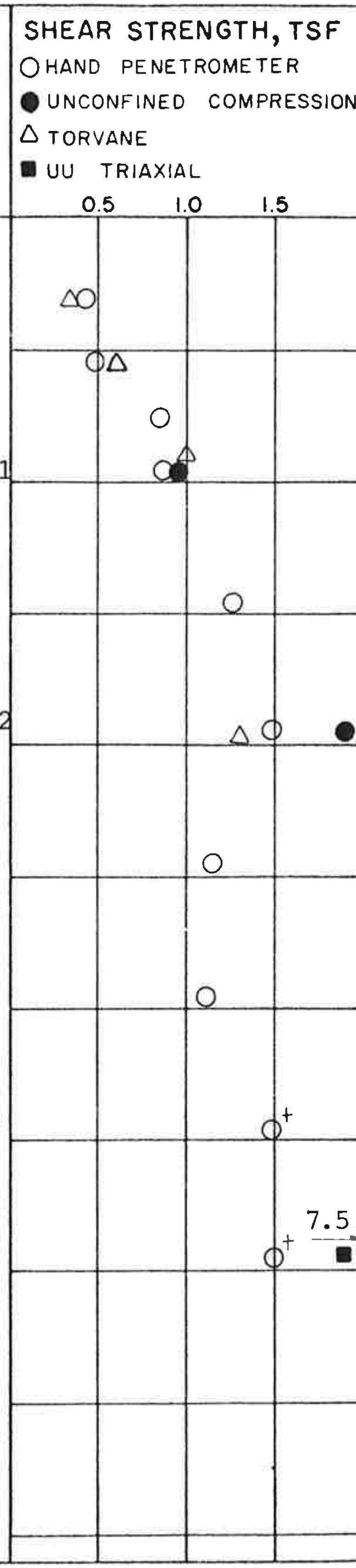
TYPE: 3-in.Shelby Tube; 2-in.Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 42'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0	Fill: firm gray clay -gravel & sand 0'-2'					
			23		56	34
5	Stiff gray clay -w/calcareous nodules -tan & light gray @ 6'		38			
			21		56	34
10			21	111		
	Very stiff tan & light gray sandy clay					
15			14		28	13
20			15	122		
	-red & light gray w/calcareous nodules & silt seams below 23'					
25			18		20	8
30			15		37	20
	Very stiff tan & light gray clay -red & light gray w/calcareous nodules @ 33'					
35			20			
	-slickensided below 38'					
40			19	112		
45						
50						

SHEAR STRENGTH, TSF



COMPLETION DEPTH: 40'
DATE: 9-4-86

DEPTH TO WATER IN BORING: 9.1'

Geotest Engineering Inc.

G-15

LOG OF BORING NO: 5C-6

LOCATION: 74'W. of ℄ of Sampson
21'N. of ℄ of Harrisburg

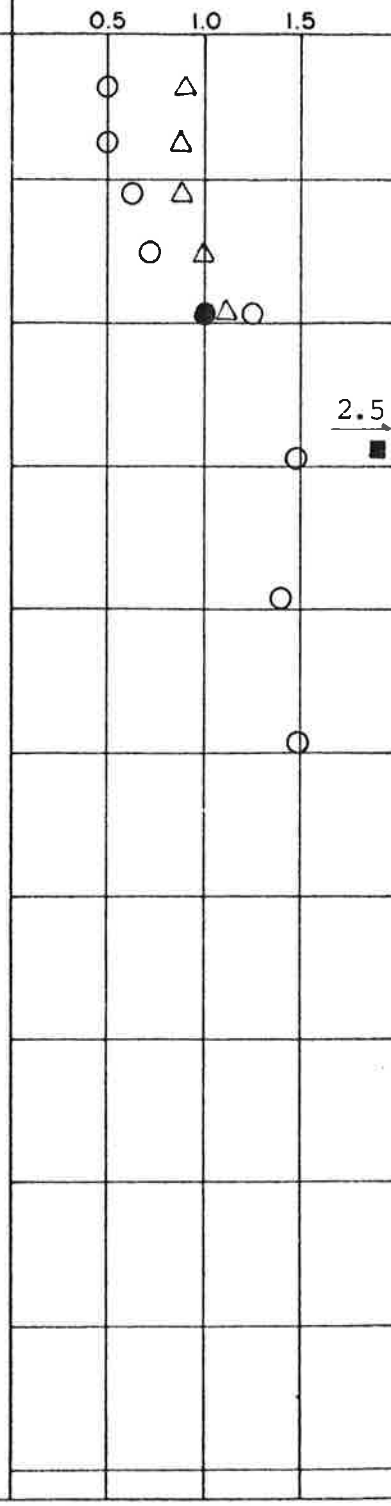
TYPE: 3-in. Shelby Tube; 2-in. Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 42'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0–5	Fill: stiff dark gray clay -asphalt 0"-2", brick 2"-5" & concrete 5"-11" -tan & light gray w/calcareous nodules below 4'		20		51	30
			20			
5–13	Stiff to very stiff gray clay		22		62	38
			25			
			25	103		
13–18	Very stiff tan & light gray sandy clay w/calcareous nodules		16	113	29	14
18–25	Very stiff red & light gray clay w/calcareous nodules -w/sand seams 18'-20'		21			
			21		63	39

SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
△ TORVANE
■ UU TRIAXIAL



COMPLETION DEPTH: 25'
DATE: 9-5-86

DEPTH TO WATER IN BORING: 9'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-7

LOCATION: 64'N. of ℄ of Harrisburg
15'W. of ℄ of Everton

TYPE: 3-in. Shelby Tube; 2-in. Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 40.2'

SHEAR STRENGTH, TSF: ○ HAND PENETROMETER; ● UNCONFINED COMPRESSION; △ TORVANE; ■ UU TRIAXIAL (scale 0.5, 1.0, 1.5)

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	SHEAR STRENGTH, TSF	LIQUID LIMIT	PLASTICITY INDEX
0	Fill: stiff gray & tan clay -asphalt 0"-3", shell fragments 3"-1' -w/ferrous nodules 1'-4'		28			75	47
5	-w/calcareous nodules below 6'		29			72	45
	Stiff to very stiff tan & light gray clay		32				
10			28	101		67	41
	-w/silt pockets @ 13'						
15			21				
20	Very stiff tan & light gray sandy clay		13	118		31	15
	-w/sand seams below 23'						
25	-sand layer 26'-27' -stiff light gray & tan @ 28'		19				
30			16				
35			13	113	2.3	22	9
	Medium dense tan silty fine sand						
40		27	20		18% Passing #200 Sieve		
45							
50							

COMPLETION DEPTH: 40'
DATE: 9-4-86

DEPTH TO WATER IN BORING: 12.0'

Geotest Engineering Inc.

PLATE A-7

LOG OF BORING NO: 5C-8

LOCATION: 224'N. of ₵ of Preston
5'E. of ₵ of Everton

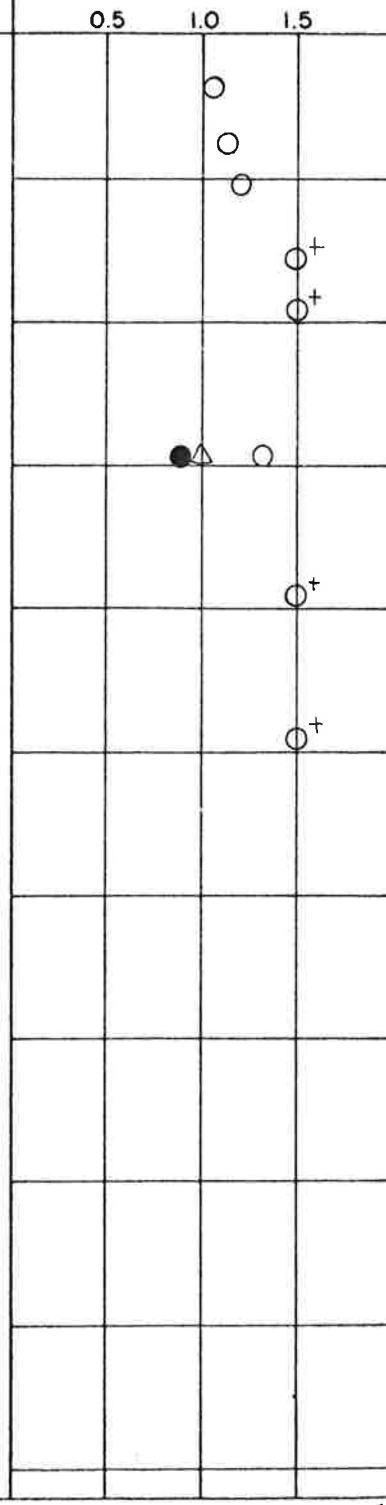
TYPE: 3-in. Shelby Tube; 2-in. Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 40.6'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0	Fill: very stiff gray clay w/ ferrous nodules		6			
	-asphalt 0"-3", shell-fragments 3"-1'		25		63	39
	-w/calcareous nodules @ 3'		30		65	40
5			33			
	Very stiff light gray clay, slickensided, w/siltstone		20			
10	-red & light gray @ 7'		25		59	36
	Very stiff light gray & tan sandy clay w/ferrous nodules & sand pockets					
15			17	121	35	18
20			16			
25			17		42	23

SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
△ TORVANE
■ UU TRIAXIAL



COMPLETION DEPTH: 25'
DATE: 8-29-86

DEPTH TO WATER IN BORING: 12'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-9

LOCATION: 15'N. of N.curb of Commerce
5'W. of ℄ of Everton

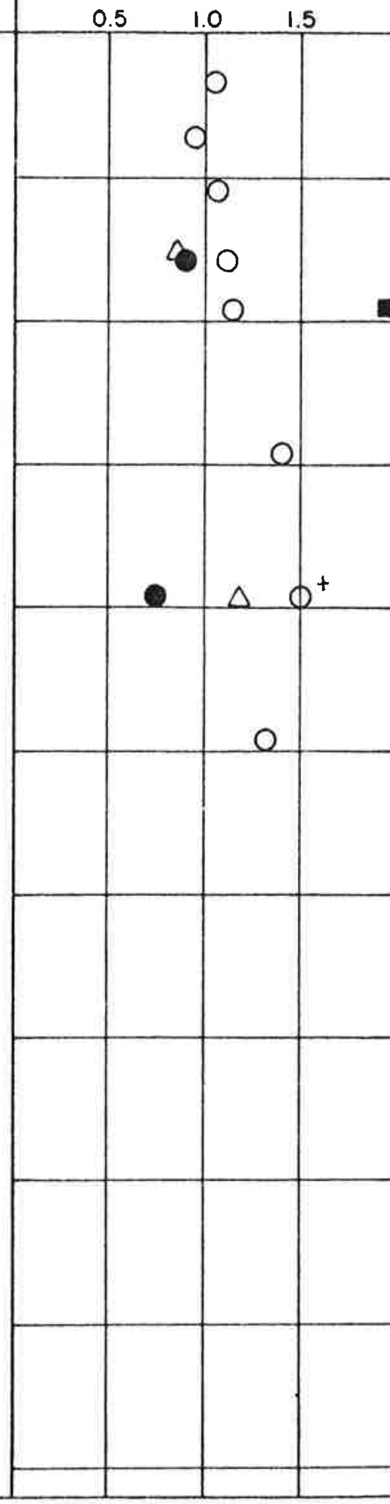
TYPE: 3-in.Shelby Tube; 2-in.Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 40'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0–5	Fill: stiff gray clay w/calcareous nodules, sand pockets & shell fragments -asphalt 0"-3", shell fragments 3"-10" -slickensided @3'		3 25 25 25		64	40
5–14	Stiff to very stiff gray clay w/ferrous & calcareous nodules -tan & light gray @ 7.5' -w/calcareous pockets & sand pockets @ 8' -light gray & tan @ 9'		20 22	106 106	72	45
14–18	Very stiff light gray & tan sandy clay w/ferrous & calcareous nodules		14		31	15
18–25	Very stiff red & light gray clay, slickensided, w/calcareous nodules		27 27	96		

SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
△ TORVANE
■ UU TRIAXIAL



COMPLETION DEPTH: 25'

DATE: 9-2-86

DEPTH TO WATER IN BORING: 5.9'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-10

LOCATION: 130'S. of ₵ of Runnels
5'W. of ₵ of Everton

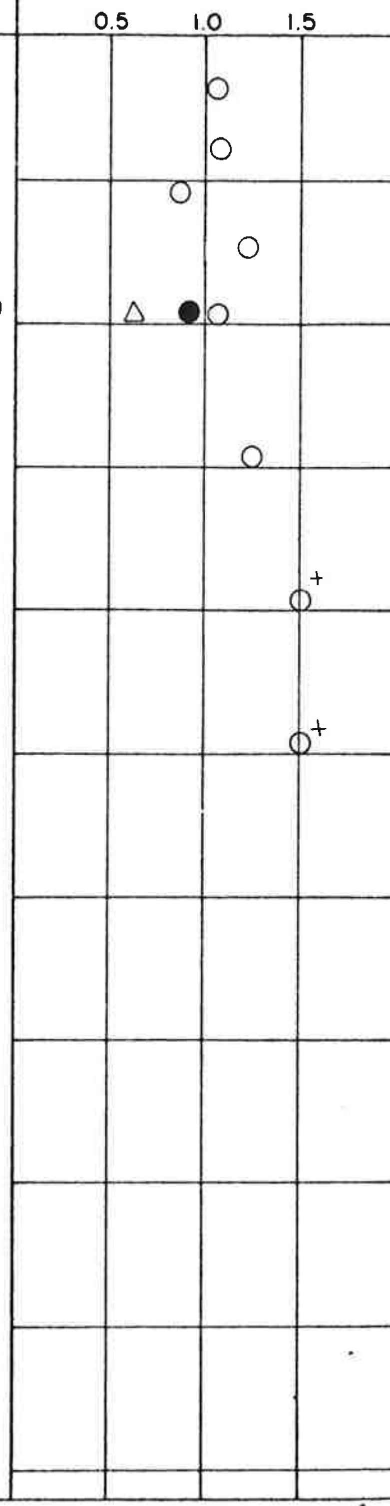
TYPE: 3-in. Shelby Tube; 2-in. Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 43'

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	LIQUID LIMIT	PLASTICITY INDEX
0–5	Fill: stiff to very stiff gray clay w/ferrous, calcareous nodules & shell fragments -asphalt 0"-3.5", stabilized shell 3.5"-10.5"		4 22 22		62	38
5–10	Stiff to very stiff light gray & tan clay, slickensided -red, slickensided, w/siltstone nodules 7'-8.5'		22 20 19	109	53	31
10–20	Stiff to very stiff red & light gray sandy clay w/calcareous nodules & clay pockets -light gray & tan below 10' -slickensided @ 18'		15 20		31	15
20–25	Very stiff red & light gray clay, slickensided		28		77	19

SHEAR STRENGTH, TSF
○ HAND PENETROMETER
● UNCONFINED COMPRESSION
△ TORVANE
■ UU TRIAXIAL



COMPLETION DEPTH: 25'
DATE: 8-29-86

DEPTH TO WATER IN BORING: 12'

Geotest Engineering Inc.

LOG OF BORING NO: 5C-11

LOCATION: 70'S. of ℄ of Navigation (E.Bound)
6'W. of ℄ of Everton

TYPE: 3-in. Shelby Tube; 2-in. Split Barrel.

PROJECT: East Water Program, Contract 5C

SURFACE ELEV. 42'

Shear strength, TSF: ○ Hand Penetrometer; ● Unconfined Compression; △ Torvane; ■ UU Triaxial (scale 0.5, 1.0, 1.5)

DEPTH, FEET	DESCRIPTION OF MATERIAL	BLOWS/FOOT	MC, %	U.D.W., PCF	SHEAR STRENGTH, TSF	LIQUID LIMIT	PLASTICITY INDEX
0	Fill: very stiff gray clay, slickensided w/calcareous nodules -asphalt 0"-3", stabilized shell 3"-1'		25			66	41
	-w/shell fragments @ 3'		25				
5	Very stiff light gray & tan clay w/sand pockets & calcareous nodules		23			64	39
			26				
10	-slickensided @ 7'		21	106		48	28
	Stiff to very stiff tan sandy clay -very sandy 11.5'-14.5'						
15			17	112			
	Very stiff red & light gray clay, slickensided w/calcareous nodules						
20			29			81	52
25			22	102	+ 4.2		
30							
35							
40							
45							
50							

COMPLETION DEPTH: 25'
DATE: 9-2-86

DEPTH TO WATER IN BORING: 9.0'

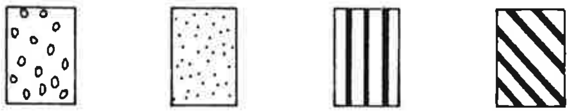
Geotest Engineering Inc.

JOB NO. 14G152

SYMBOLS AND TERMS USED ON BORING LOGS

SOIL TYPES

(SHOWN IN SYMBOL COLUMN)

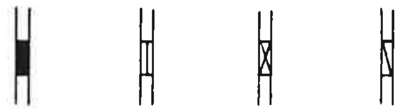


Gravel Sand Silt Clay

Predominant type shown heavy

SAMPLER TYPES

(SHOWN IN SAMPLES COLUMN)



Shelby Tube Piston Split Spoon No Recovery

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (Major Portion Retained on No.200 Sieve): Includes (1) clean gravels and sands, and (2) silty or clayey gravels and sands. Condition is rated according to relative density, as determined by laboratory tests.

Descriptive Term	Standard Penetration, Resistance, Blows/Ft.	Relative Density
Loose	0 - 10	0 to 40%
Medium dense	10 - 30	40 to 70%
Dense	30 - 50	70 to 100%

FINE GRAINED SOILS (Major portion passing No. 200 sieve): Includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by unconfined compression tests.

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH TONS / Sq. Ft.
Very soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	4.00 and higher

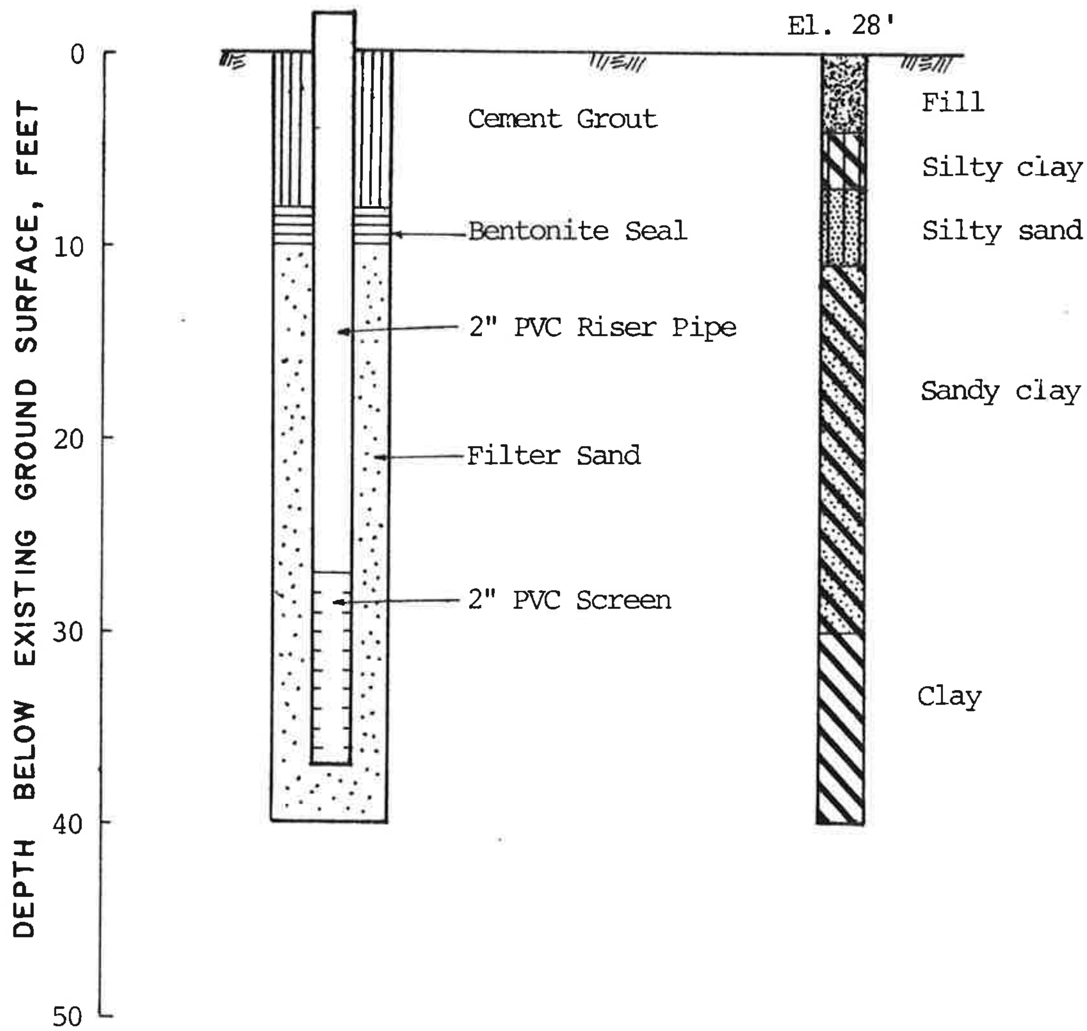
Note: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above, because of planes of weakness or cracks in the soil. The consistency ratings of such soils are based on penetrometer readings.

TERMS CHARACTERIZING SOIL STRUCTURE

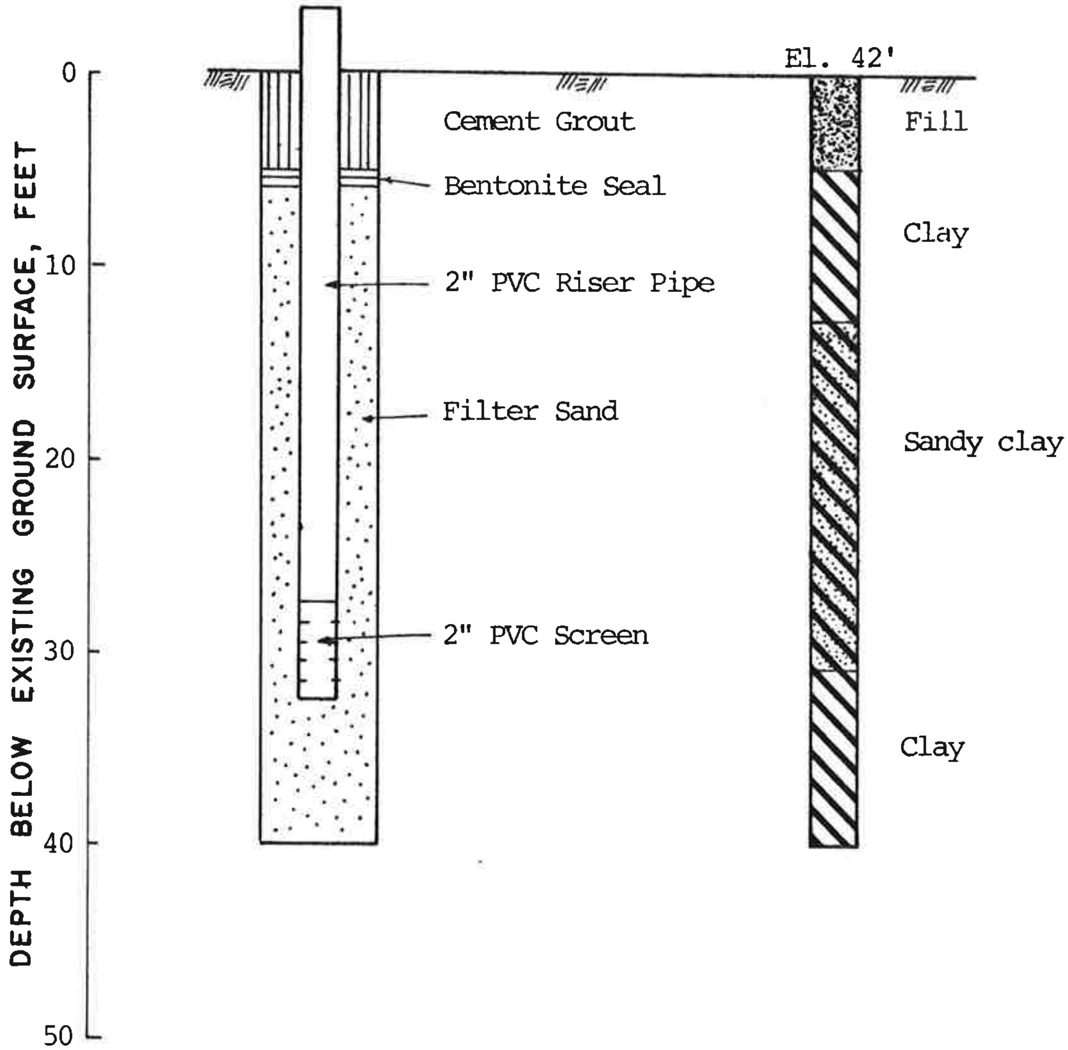
Parting: - paper thin in size   Seam: - 1/8"-3" thick   Layer: - greater than 3"

Slickensided	- having inclined planes of weakness that are slick and glossy in appearance.
Fissured	- containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
Laminated	- composed of thin layers of varying color and texture.
Interbedded	- composed of alternate layers of different soil types.
Calcareous	- containing appreciable quantities of calcium carbonate.
Well graded	- having wide range in grain sizes and substantial amounts of all intermediate particle sizes.
Poorly graded	- predominantly of one grain size, or having a range of sizes with some intermediate size missing.
Flocculated	- pertaining to cohesive soils that exhibit a loose knit or flakey structure.

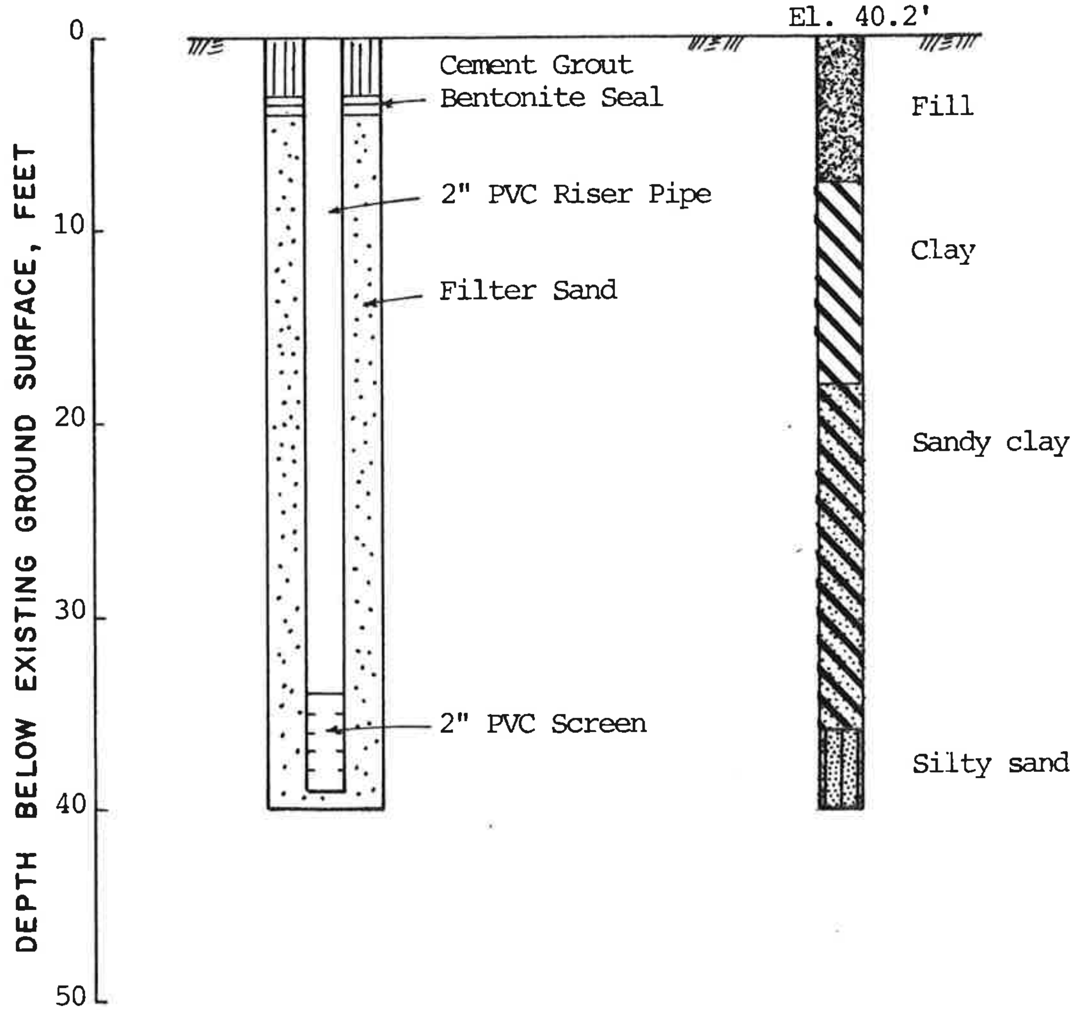
Geotest Engineering, Inc.



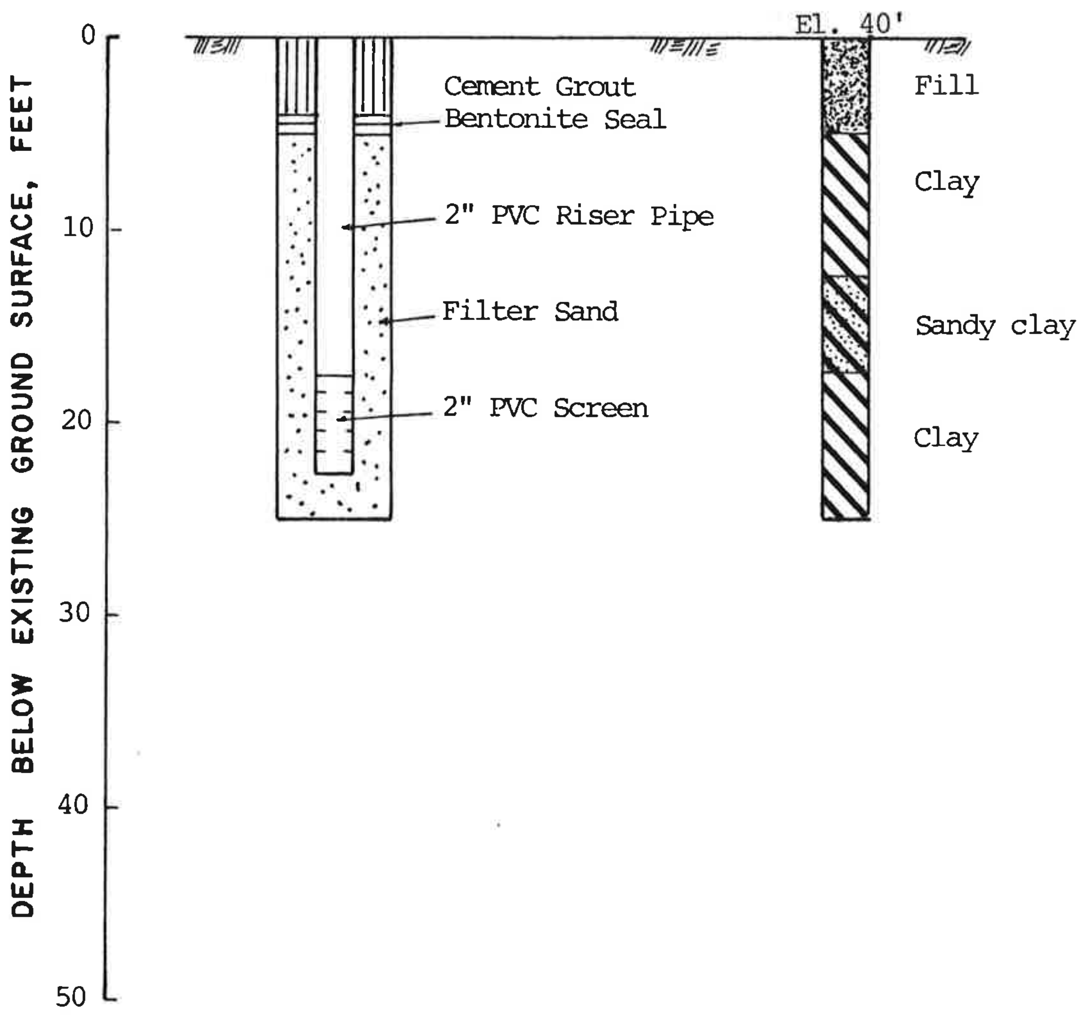
SCHEMATIC AND SOIL PROFILE
PIEZOMETER 5C-3P



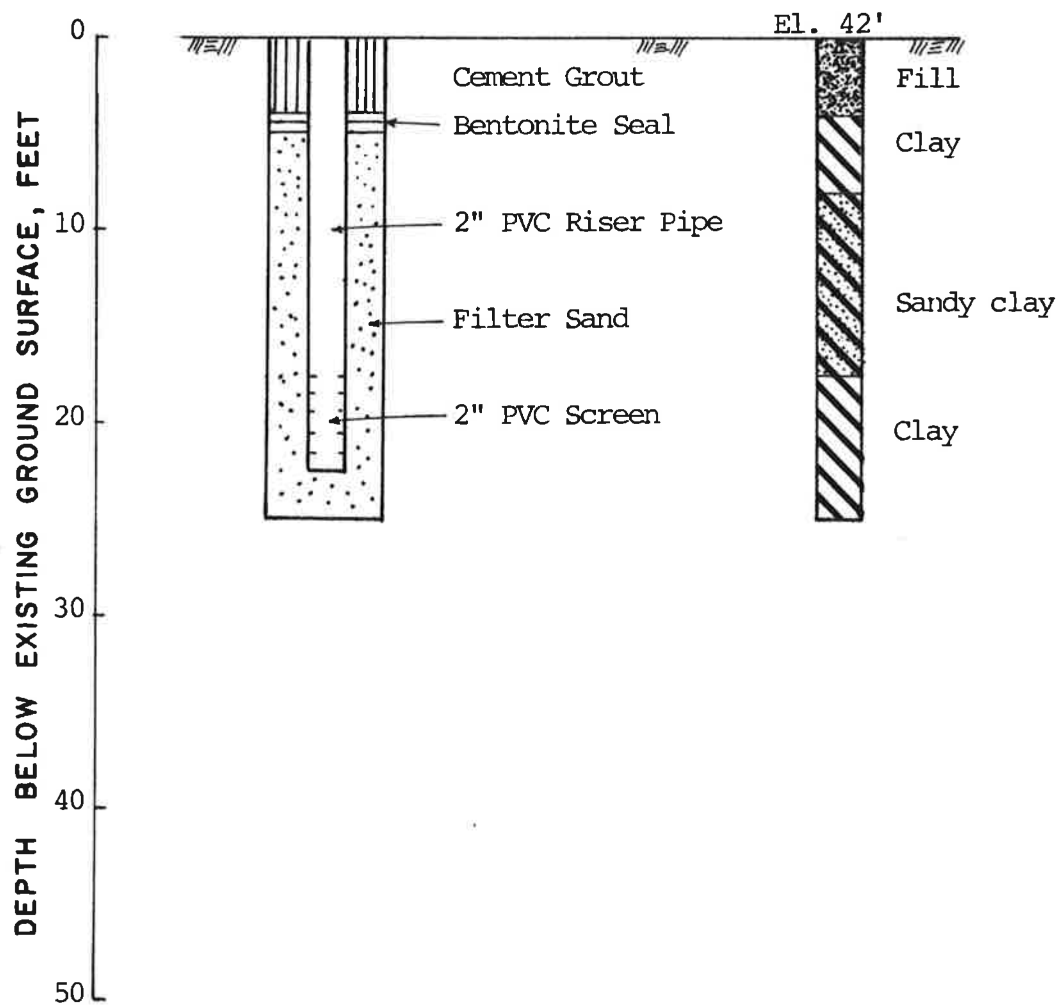
SCHEMATIC AND SOIL PROFILE
PIEZOMETER 5C-5P



SCHEMATIC AND SOIL PROFILE
PIEZOMETER 5C-7P



SCHEMATIC AND SOIL PROFILE
PIEZOMETER 5C-9P



SCHEMATIC AND SOIL PROFILE

PIEZOMETER 5C-11P

JOB # : 86-1005
DATE : 5-12-86
LOCATION : C-1
FILE # : 2

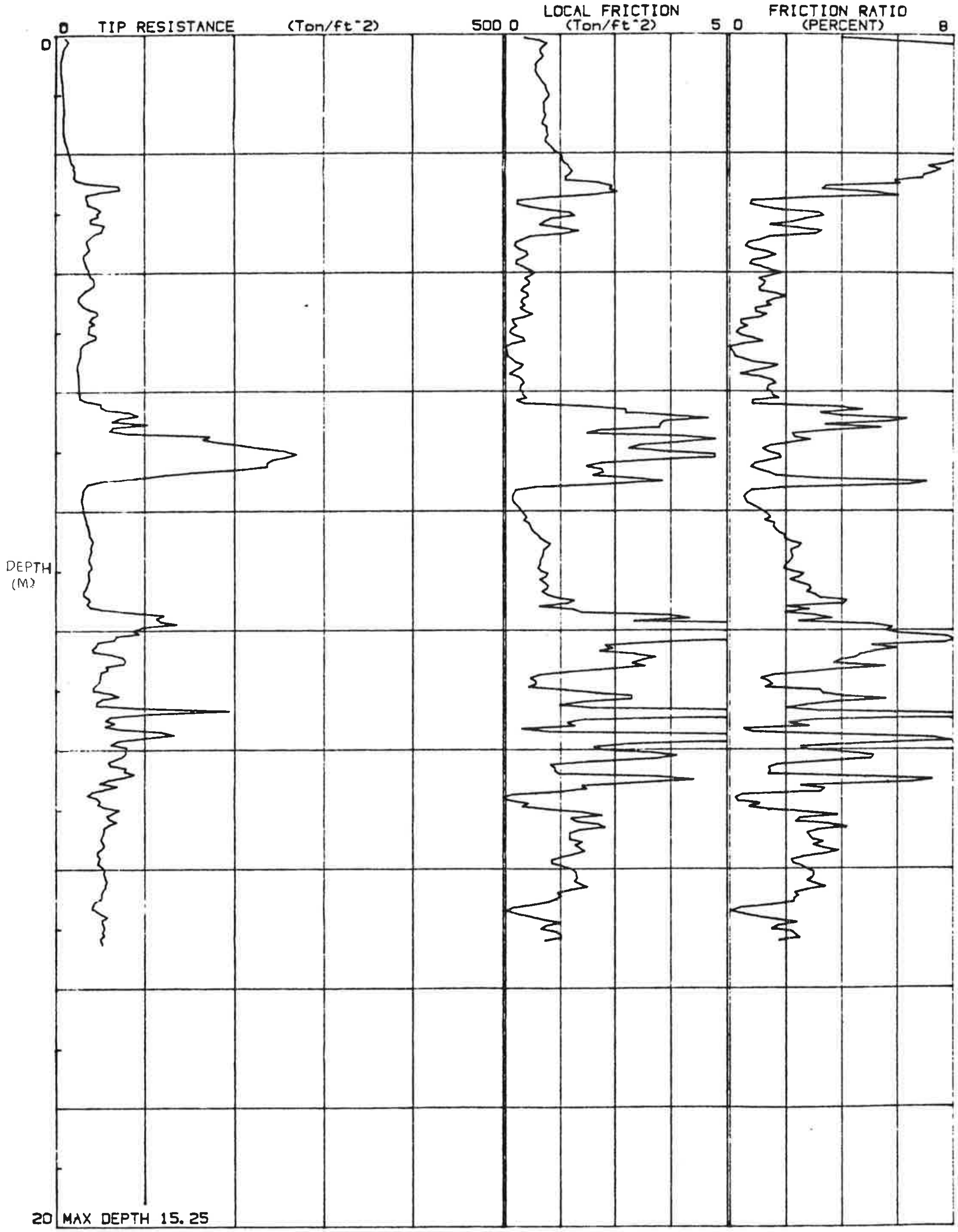


PLATE A-18

JOB # : 86-1005
DATE : 5-12-86
LOCATION : C-1
FILE # : 2

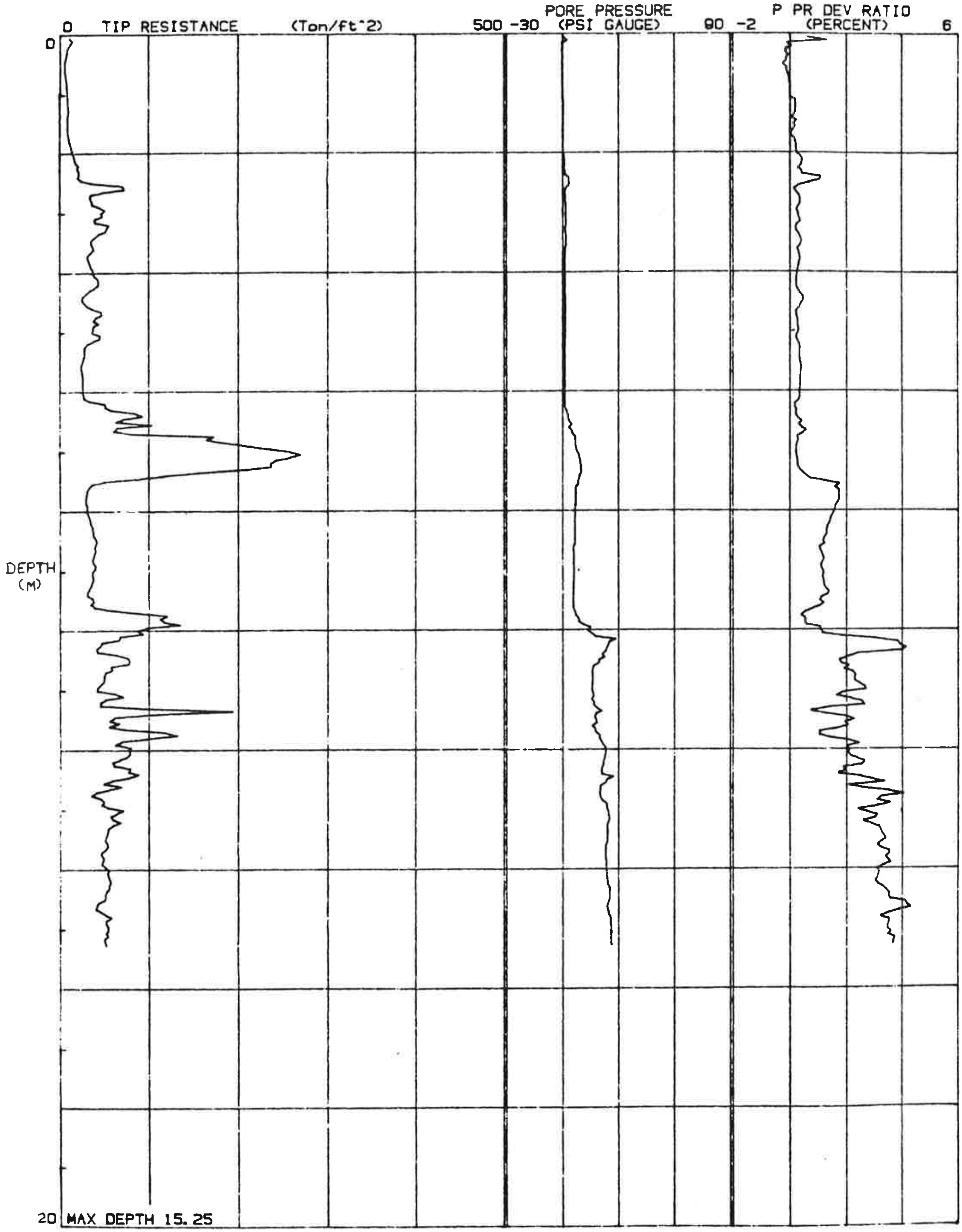
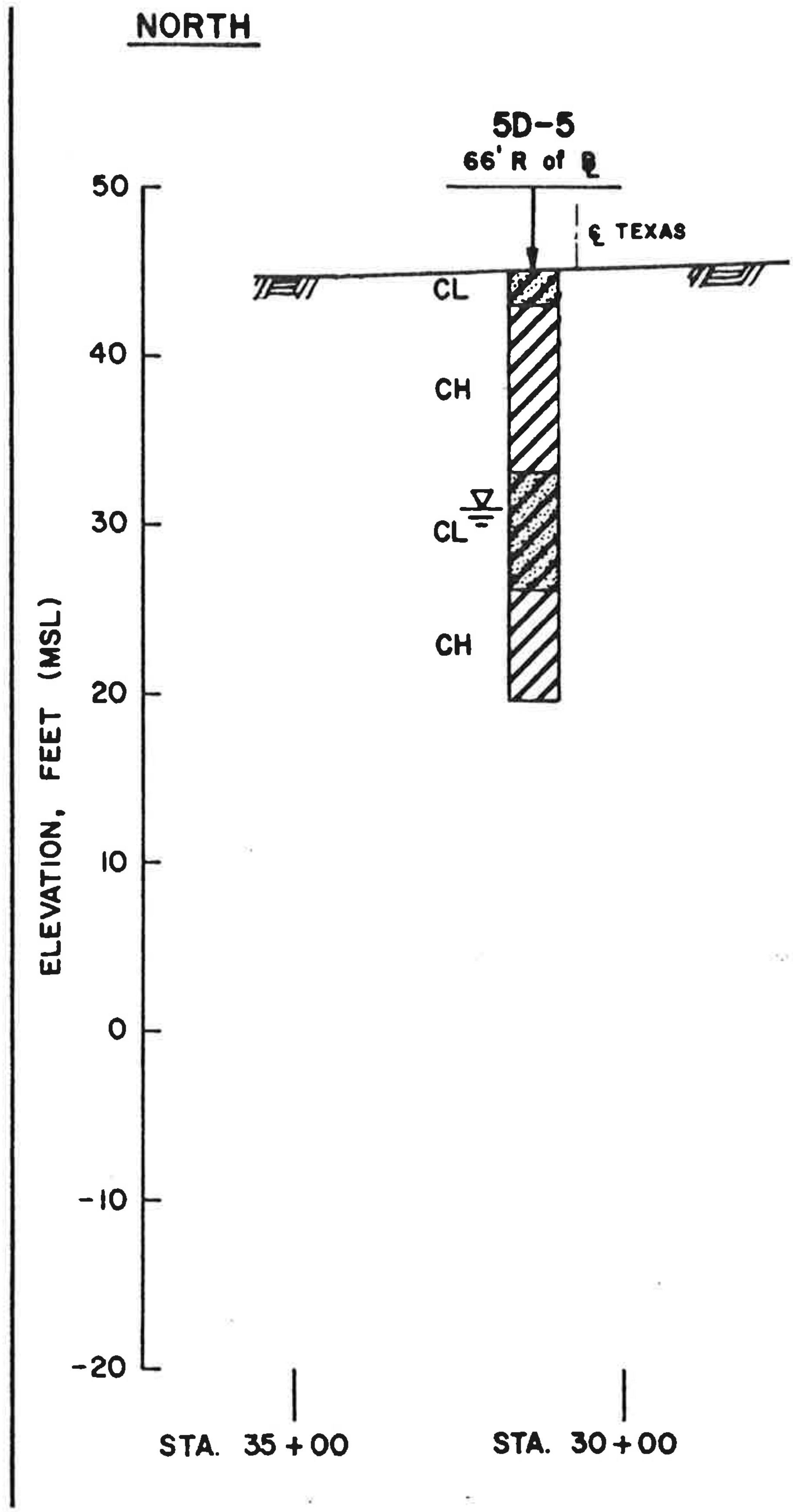


PLATE A-19



NORTH
5D-5
66' R of ℄
℄ TEXAS
50
40
30
20
10
0
-10
-20
ELEVATION, FEET (MSL)
CL
CH
CL
CH
STA. 35+00
STA. 30+00

Job No.: 14G152

murillo engineering, incorporated

10910 STANCLIFF ROAD • (713) 933-9702 • HOUSTON TEXAS 77099

PROJECT CITY OF HOUSTON-EAST WATER PROGRAM-PACKAGE 5B **BORING** 5B-13

DATE 4-15-86 **TYPE** 3" Core **LOCATION** Sta.1+31;34' R of BL

DEPTH IN FEET	DESCRIPTION	BLOWS / FOOT	M.C., %	U.D.W., PCF	LIQUID LIMIT	PLASTIC LIMIT
0	Dark gray sandy clay; FILL					
	Dark gray clay		26	99	60	22
5	- brown and gray with calcareous nodules		23	106		
10						
	Gray and tan sandy clay with clay pockets					
15						
	Red and gray clay with slickensides		27	99	67	22
20						
25						
	Red and gray silty clay with clay partings		22	106	25	16
30						
	Tan and gray clay					
35	- red and gray with calcareous nodules and slickensides		19	110	53	22
40	BOTTOM AT 40 FEET					

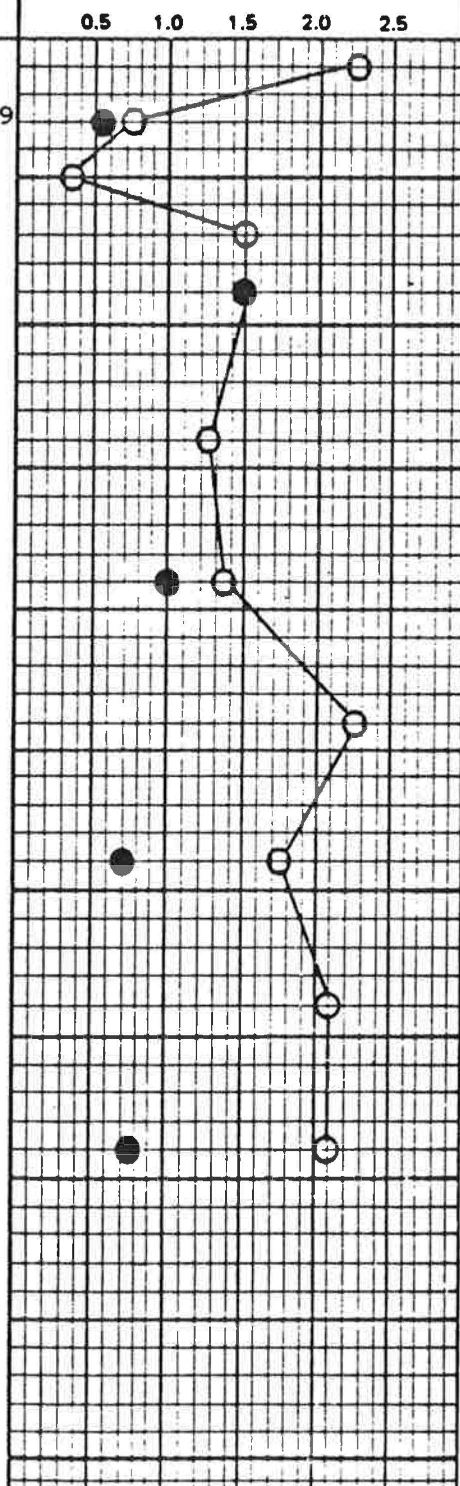
BOTTOM AT 40 FEET

1. Bailed boring to 26'-2" upon completion
2. Water level at 12'-11" after 24 hours

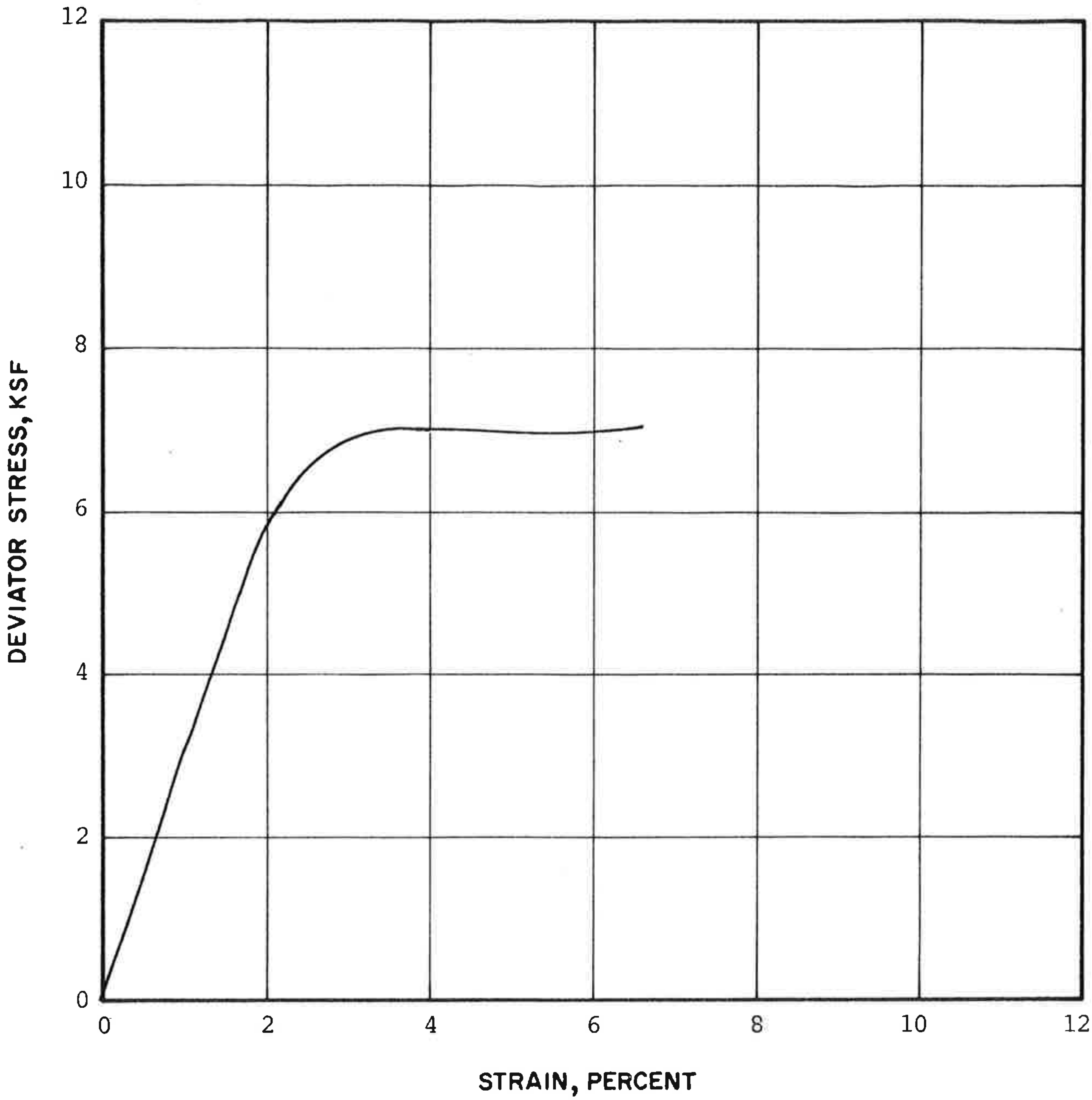
SHEAR STRENGTH, TSF

O – POCKET PENETROMETER

● – LABORATORY UNCONFINED



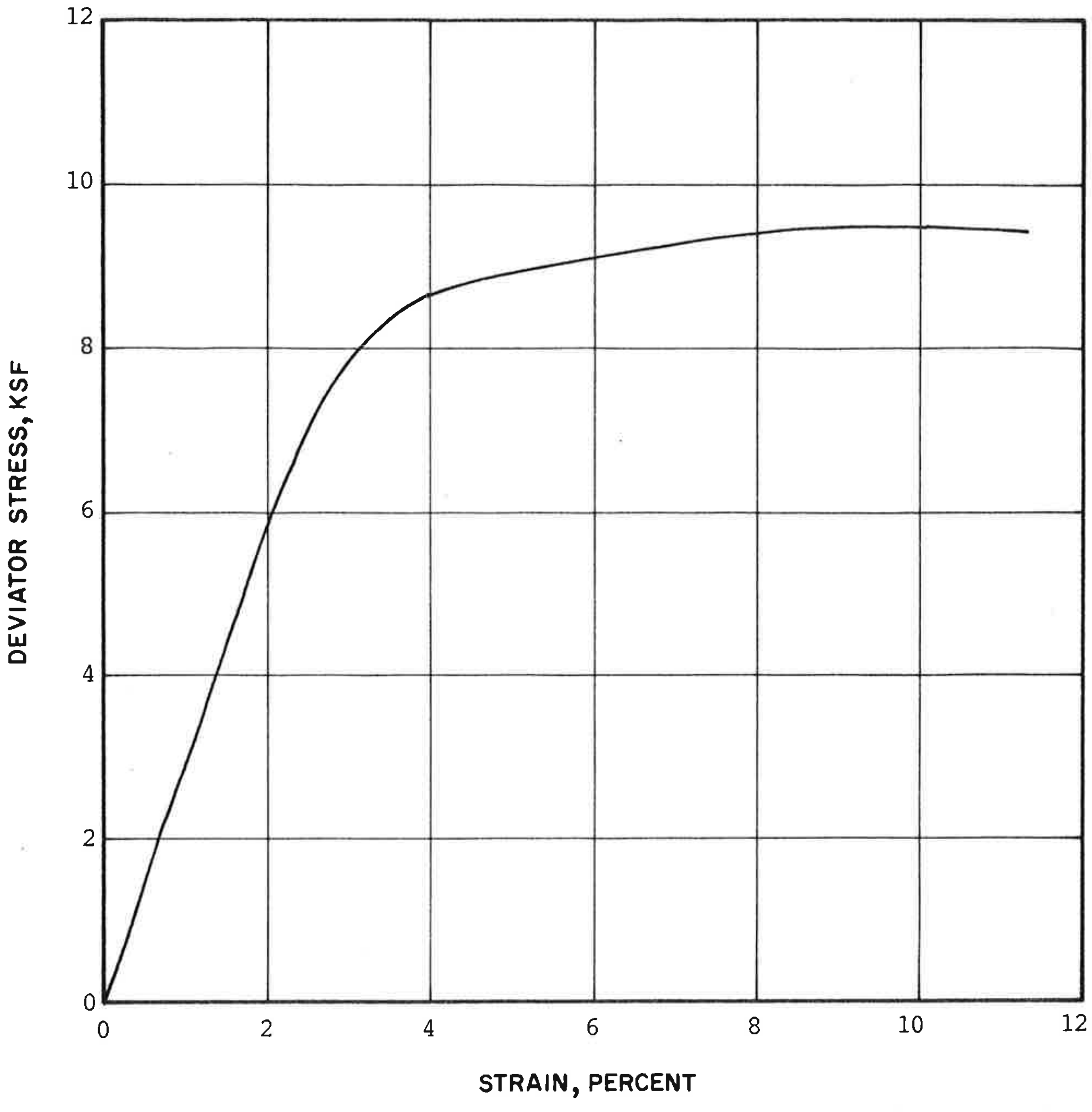
APPENDIX B



BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-2	8	23-25	2.88	Red and light gray clay

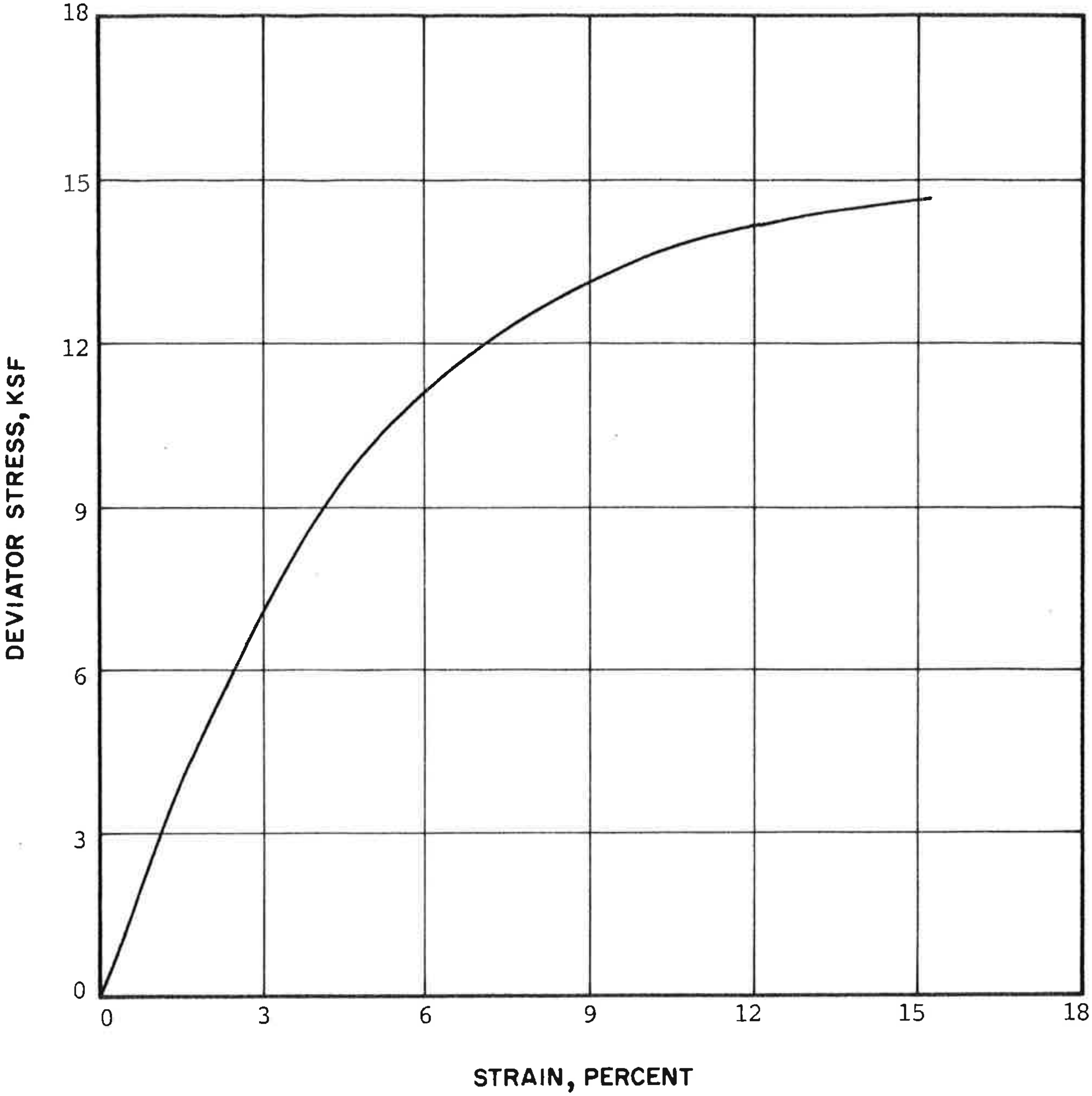
STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED

Geotest Engineering Inc.



BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-4	5	8-10	1.80	Brown, red and light gray clay and sandy clay

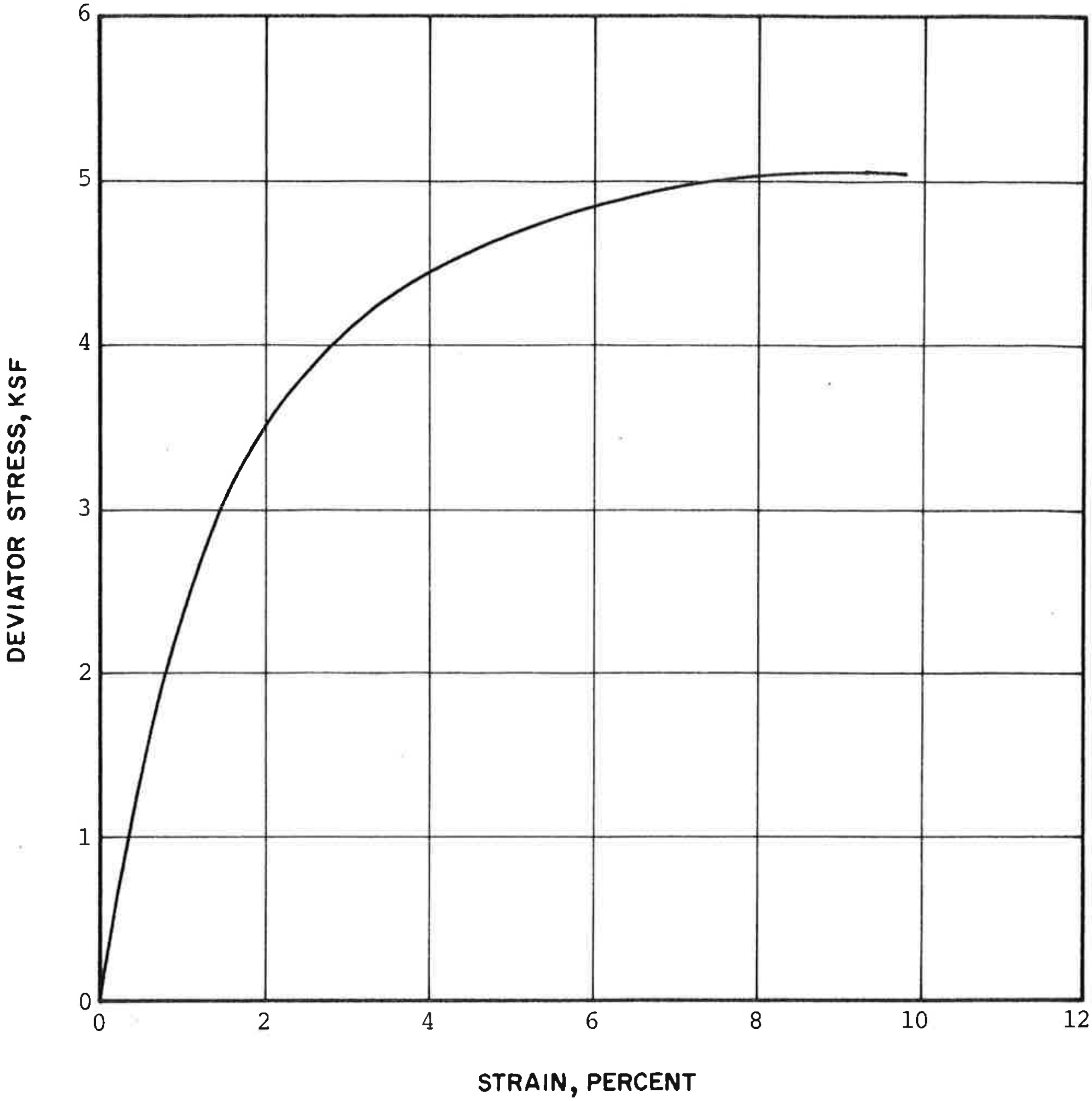
STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED



BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-5	10	38-40	5.04	Tan and light gray clay, slickensided

STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED

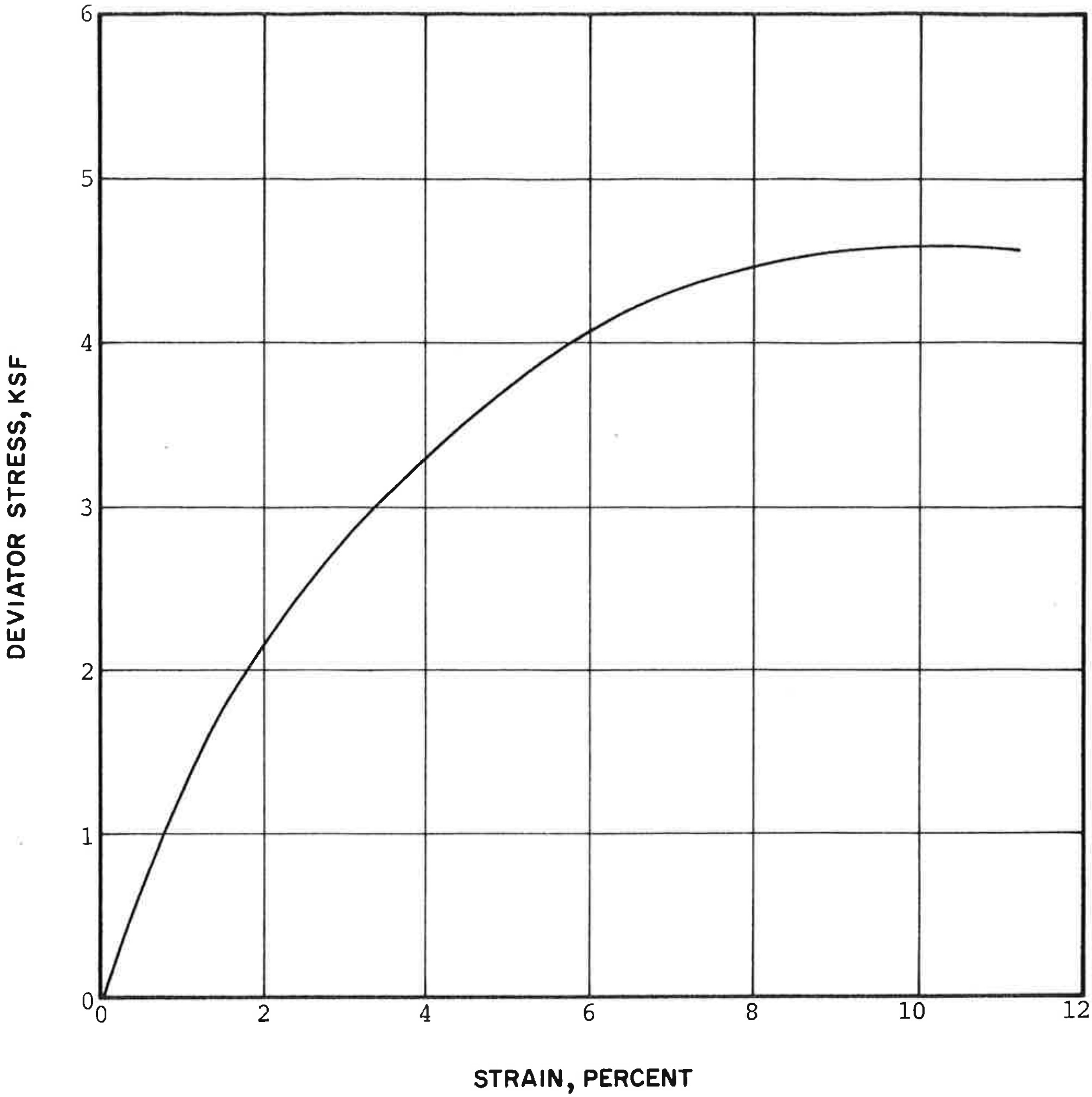
Geotest Engineering Inc.



BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-6	6	13-15	2.16	Tan and light gray sandy clay

STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED

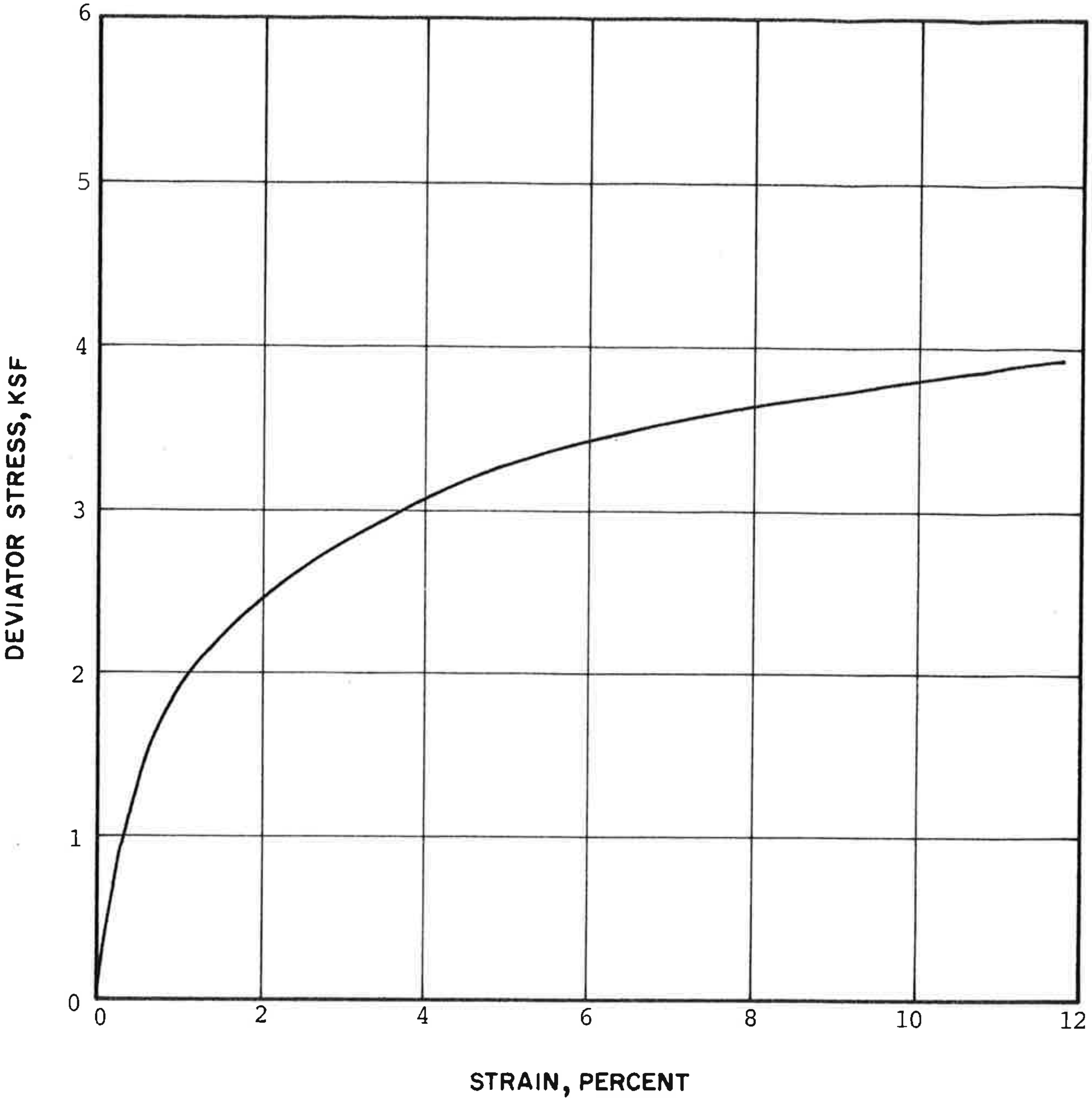
Geotest Engineering Inc.



BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-7	9	33-35	4.32	Light gray and tan sandy clay

STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED

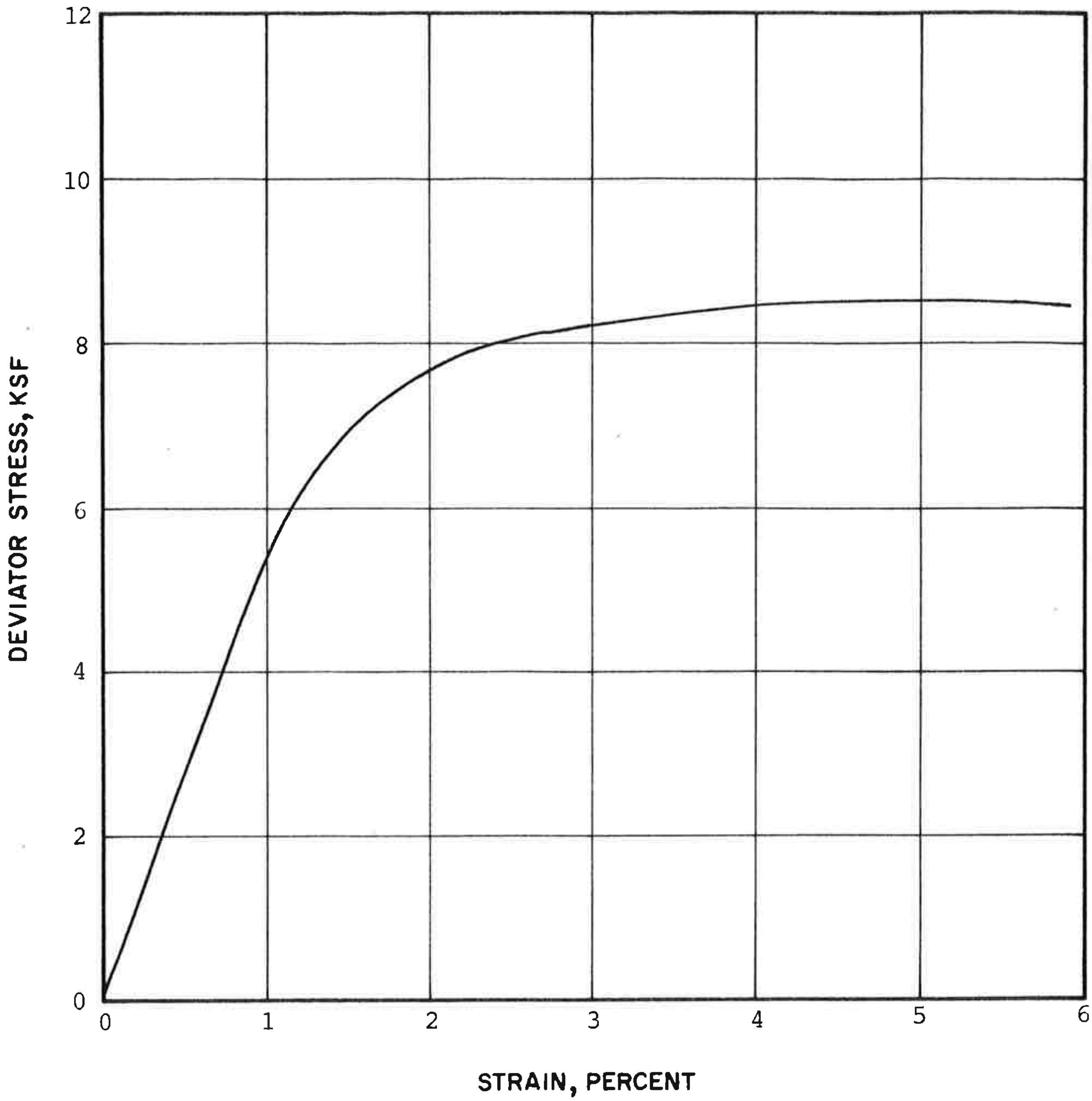
Geotest Engineering Inc.



BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-9	6	8-10	2.88	Light gray and tan clay

STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED

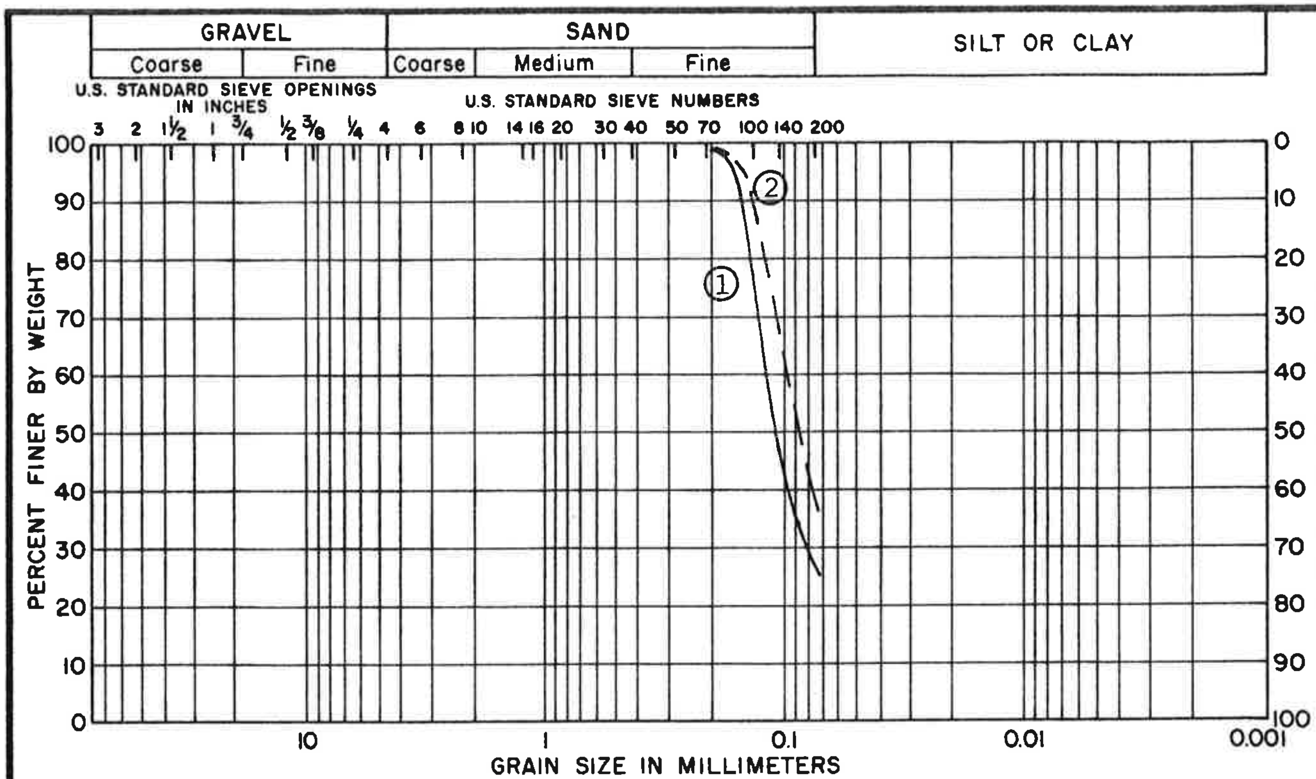
Geotest Engineering Inc.



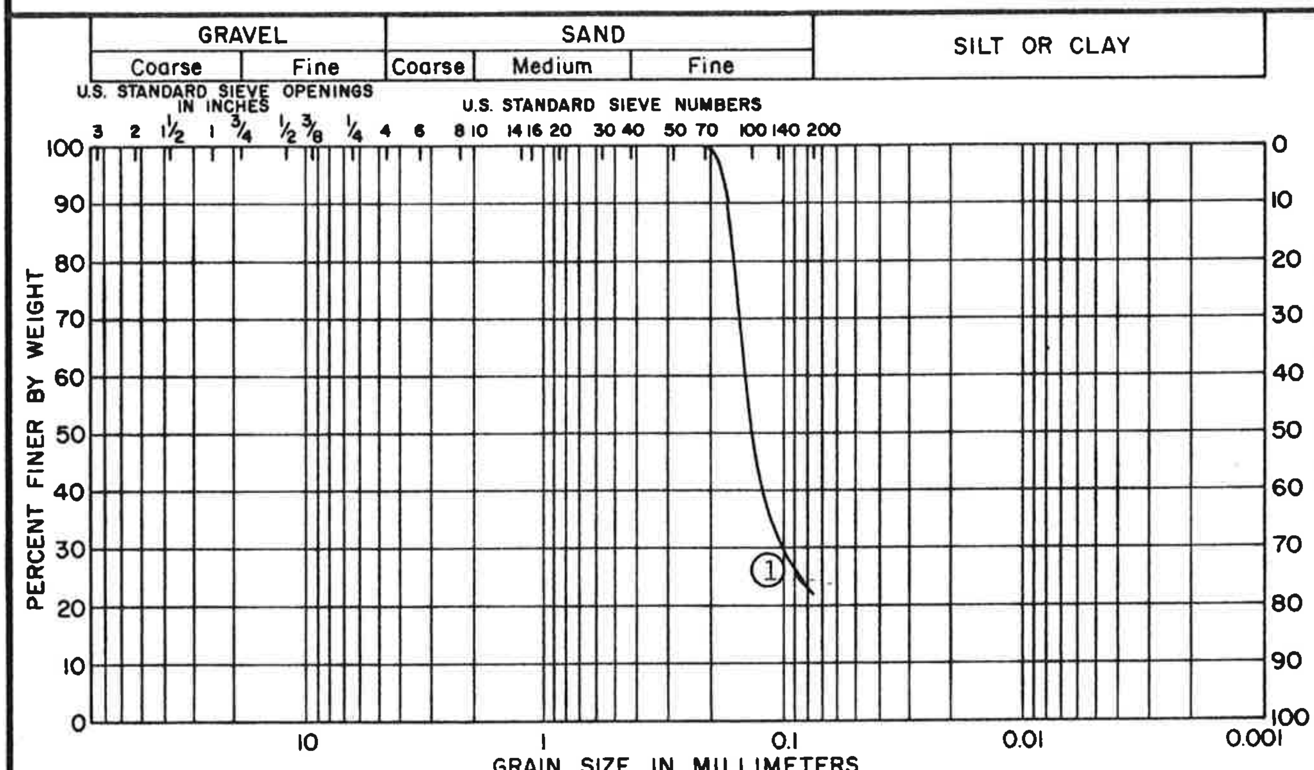
BORING NO.	SAMPLE NO.	DEPTH, FT.	CONFINING PRESSURE, TSF	MATERIAL
5C-11	8	23-25	2.88	Red and light gray clay, slickensided

STRESS-STRAIN CURVES
TRIAXIAL COMPRESSION TEST
UNCONSOLIDATED UNDRAINED

Geotest Engineering Inc.



Curve No.	Boring No.	Depth (ft.)	Description
1	5C-1	33' - 35'	Tan silty fine sand
2	5C-3	8' - 10'	Light gray silty fine sand



Curve No.	Boring No.	Depth (ft.)	Description
1	5C-7	38' - 40'	Tan silty fine sand

GRAIN SIZE DISTRIBUTION CURVES

FORM NO. GE 308
JOB NO.

APPENDIX C

VERTICAL EARTH LOAD ON RIGID DITCH CONDUIT

$$Wc = Cd.r.Bd.Bd$$

Where
- Wc = vertical load per unit length of conduit,
- Cd = load coefficient,
- r = wet unit weight of backfill material, and
- Bd = horizontal width of trench at top of conduit.

Granular Backfill:
- use Ku'= 0.1924
- r = 125 pcf

MAXIMUM BACKFILL LOADS ON TRENCH CONDUITS, Wc (lbs/lin ft.)

H (feet)	Width of Trench at Top of Conduit (feet)				
	Bd=	9	10	11	12
6	Wc=	5954	6697	7442	8187
7	Wc=	6806	7671	8537	9405
8	Wc=	7622	8607	9595	10584
9	Wc=	8405	9509	10616	11727
10	Wc=	9154	10376	11603	12833

Notes: 1. Surface loads not included,
2. H = Depth of fill to top of conduit.

SAMPLE CALCULATION:

For a rigid ditch conduit with an I.D. of 84 in.
The width of trench at top of conduit is 11 feet.
The conduit is covered with 6 feet of granular backfill.
The wet unit weight of the backfill is 125 pcf.

Therefore, H = 6 feet
r = 125 pcf
Bd= 11 feet
and H/Bd= 0.55
for granular fill Ku'= 0.1924 ,
From the curve in Plate 8, find Cd = 0.49
Substituting in equation:

$$Wc = Cd.r.Bd.Bd$$

and find

$$Wc = 0.49 \times 125 \times 11 \times 11$$
$$= 7442 \text{ lbs/linear ft.}$$

LOAD ON CONDUIT DUE TO TRAFFIC LOAD

$$Wt = \frac{1}{A} Ic.Ct.P$$

Where
- Wt = average load per unit length of conduit due to wheel load ;
- A = effective length of conduit section on which load is computed (recommended 3 ft.) ;
- Ic = impact factor = 1.5 ;
- Ct = load coefficient ; and
- P = concentrated wheel load on surface (use 16000 lbs for dual-tired wheel load).

TRAFFIC LOADS ON TRANCH CONDUITS, Wt (lbs/lin ft.)

D (in.)	H (ft.)	Ct	Wt
84	6	0.230	1840
84	7	0.181	1448
84	8	0.149	1192
84	9	0.125	1000
84	10	0.102	816

Notes: 1. D = Inside diameter of pipe,
2. H = Depth of fill to top of conduit.

** SAMPLE CALCULATION FOR LOAD ON CONDUIT DUE TO TRAFFIC LOAD **

For a rigid ditch conduit with an I.D. of 84 in.
The conduit is covered with 8 feet of granular backfill.
The wheel load is 16000 lbs.
The effective length is 3 feet.
The impact factor is 1.5.

Therefore, P = 16000 lbs
Ic= 1.5
A = 3 feet
and for D = 84 in., H = 8 ft.
Ct= 0.149

find $Wt = \frac{1}{A} Ic.Ct.P = (1/3) \times 1.5 \times 0.149 \times 16000$

$= 1192$ (lbs/lin. ft)

THREE EDGE BEARING STRENGTH OF RIGID PIPE

$$Seb = \frac{(Wc + Wt) \times Fs}{Lf}$$

Where

Wc, Wt = maximum load on conduit from overburden and traffic, respectively;
Seb= three edge bearing load;
Lf = load factor for bedding classification (recommended value of Lf = 1.5);
Fs = factor of safety (recommended value of Fs = 1.2).

SAMPLE CALCULATION FOR THREE EDGE BEARING LOAD

For a reinforced concrete pipe with an I.D. of 84 in.
The width of the trench at the top of the pip is 10 feet.
The pipe is covered with 8 feet of granular backfill.

Therefore,	D =	84 inches
	Bd=	10 feet
	H =	8 feet
	Lf=	1.5
	Fs=	1.2
From Plate C-1 find	Wc=	8607 lbs/lin ft.
From Plate C-2 find	Wt=	1192 lbs/lin ft.

and find $$Seb = \frac{(Wc + Wt) \times Fs}{Lf}$$

$$= \frac{(8607 + 1192) \times 1.2}{1.5}$$

$$= 7839 \text{ lbs/lin ft.}$$

THRUST FORCES ACTING ON A BEND

$$T = 2\ P\ A\ \mathrm{Sin}\ 0/2$$
$$T_x = P\ A\ (1 - \mathrm{Cos}\ \theta)$$
$$T_y = P\ A\ \mathrm{Sin}\ \theta$$

Where
- T = resultant thrust force
- T_x = X thrust force component
- T_y = Y thrust force component
- P = maximum sustained pressure
- A = pipe cross-sectional area
- θ = bend angle

D (in.)	P (psi)	θ (deg.)	T (kips)	Tx (kips)	Ty (kips)
84	210	90	1645.8	1163.8	1163.8
84	210	73	1384.5	823.5	1112.9
84	210	55	1074.7	496.3	953.3
84	210	38	757.8	246.7	716.5
84	210	10	202.9	17.7	202.1

SAMPLE CALCULATION:

For a 55 degree horizontal bend in a 84 in. I.D. pipe using surge pressure 210 psi.

Therefore,
P = 210 psi
D = 84 in.
θ = 55 degree

and find
$$A = \pi . D^2/4 = \pi \times (84^2/4) = 5541.8\ (\mathrm{in.}^2)$$

$$T = 2.P.A.\mathrm{Sin}(\theta/2) = 2 \times 210 \times 5541.8 \times \mathrm{Sin}(55^\circ/2)$$
$= 1074740$ (lbs)
$= 1074.7$ (kips)

$$T_x = P.A.(1-\mathrm{Cos}\theta) = 210 \times 5541.8 \times (1 - \mathrm{Cos}55^\circ)$$
$= 496260$ (lbs)
$= 496.3$ (kips)

$$T_y = P.A.\mathrm{Sin}\theta = 210 \times 5541.8 \times \mathrm{Sin}55^\circ$$
$= 953306$ (lbs)
$= 953.3$ (kips)

BEARING THRUST BLOCK

For $h = \frac{1}{2} H_T$

$$b = \frac{Sf.2.P.A.\sin(\theta/2)}{(3/8)r.H_T^2.Kp + C.H_T\sqrt{Kp}}$$

Where

h = height of thrust block ($h \geq Bc$);
H_T = depth to bottom of thrust block;
b = width of thrust block ($h \leq b \leq 2h$);
Sf = factor of safety = 1.5;
P = maximum sustained pressure;
A = pipe cross-sectional area,
= $(\pi D^2/4)$;
D = inside diameter of pipe;
Bc = maximum outside diameter of pipe;
θ = bend angle;
Kp = passive earth pressure coefficient and
= $\tan^2(45^\circ + \phi/2)$;
C = soil cohesion;
ϕ = soil internal friction angle.

Design Parameters:

	ϕ =	0	degree	
	Kp=	1		
	r =	125	pcf	(N.G. to 6 feet)
	r'=	62.5	pcf	(below 6 feet)
	C =	2000	psf	
	P =	210	psi	
	D =	84	inches	
	Bc=	99	inches	
and	A =	5541.8	in.^{2}	
	Sf=	1.5		

θ (deg)	H (ft)	H_T (ft)	h (ft)	b (ft)
10	6	16.5	8.3	8.3
10	7	16.5	8.3	8.3
10	8	16.5	8.3	8.3
10	9	17.5	8.8	8.8
10	10	18.8	9.4	9.4

Notes: 1. H = Depth of fill to top of pipe.
2. Bearing thrust block is not feasible at the locations where the 84-in. I.D. pipe has a bend greater than 23 deg. for 6 ft. cover and 32 deg. for 10 ft. cover (The dimension of the bearing thrust block cannot be rationally adjusted for the corresponding depth and height of the block).

SAMPLE CALCULATION FOR BEARING THRUST BLOCK

Design Parameters:

ϕ = 0 degree
Kp = 1
r = 125 pcf (N.G. to 6 feet)
r' = 62.5 pcf (below 6 feet)
C = 2000 psf
P = 210 psi
θ = 10 degree
D = 84 inches
Bc = 99 inches
and A = 5541.8 $in.^2$
Sf = 1.5
H = 8 feet

for $h = 1/2\ H_T$,

$H_T = H + Bc/2 + h/2 = H + Bc/2 + (1/2\ H_T)/2$

$= H + Bc/2 + H_T/4$

therefore $H_T = 4/3\ (\ H + Bc/2\ )$

$= 4/3\ (\ 8 + (99/12)/2\ )$

$= 16.2$ (feet)

$h = 1/2\ H_T = 1/2\ (\ 16.2\ )$

$= 8.1$ (feet) $<$ Bc (N.G.)

use $h = Bc = 99/12 = 8.25$ (feet)

and $H_T = 2h = 2 \times 8.25 = 16.5$ (feet)

equivalent $r = (125\text{x}6+62.5\text{x}(16.5-6))/16.5 = 85.2$ (pcf)

$$b = \frac{Sf.2.P.A.\sin(\theta/2)}{3/8.r.H_T^2.Kp + C.H_T\sqrt{Kp}}$$

$$= \frac{1.5 \times 2 \times 210 \times 5541.8 \times \sin(10^\circ/2)}{(3/8) \times 85.2 \times 16.5^2 \times 1 + 2000 \times 16.5 \times \sqrt{1}}$$

$= 7.3$ (feet) $<$ h (N.G.)

therefore use $b = h = 8.25$ (feet)

==

LENGTH OF JOINT RESTRAINED PIPE

$$L = \frac{Sf.K.P.A}{K.Fs+Bc.Pp}$$

Where L = restrained pipe length on each side of pipe
Sf= factor of safety (usually 1.25)
K = bend coefficient = 4 tan $\theta/2$
P = maximum sustained pressure
A = pipe cross-sectional area
Fs= unit conduit frictional resistance = Ap.f + W.tanδ
Bc= outside diameter of the conduit
Pp= passive soil pressure = r_s .Hc.Kp+2.Cs.$\sqrt{Kp}$
θ = bend angle
Ap= conduit surface area per unit length = $(\pi Bc)/2$, (assume 1/2 the pipe circumference bears against the backfill soil)
f = unit cohesion between conduit and backfill, = 0.5 Cb
W = unit normal force on the pipe = $\pi . r_b$ .H.Bc.R
δ = frictional angle between conduit and backfill, = 0.75 ϕb
r_b= backfill unit weight
r_s= in-situ soil unit weight
Hc= mean depth from ground surface to the plane of of resistance (center line of a pipe)
Kp= passive earth pressure coefficient, = $\tan^2(45^\circ + \phi s/2)$
Cb= backfill cohesion
Cs= in-situ soil cohesion
ϕb= backfill soil internal friction angle
ϕs= in-situ soil internal friction angle
R = reduction factor based on trench condition (generally 2/3)

CASE:

Maximum sustained pressure, P = 210 psi

Factor of safety, Sf = 1.25

Condition I. Groundwater level at 6 feet below ground surface

Condition II. Groundwater at ground surface(during heavy flood)

Condition I. Groudwater Level at 6 ft. below Ground Surface

Maximum Sustained Pressure, P = 210 psi

Factor of Safety, Sf = 1.25

Trench Backfill Parameters:

r_b = 120 pcf (N.G. to 6 feet)
r_b' = 60 pcf (below 6 feet)
ϕb= 25 degree
Cb= 0 psf
δ = 0.75 ϕb degree
f = 0.5 Cb psf

In-Situ Soil Parameters:

r_s = 125 pcf (N.G. to 6 feet)
r_s' = 62.5 pcf (below 6 feet)
ϕs= 0 degree
Cs= 2000 psf
Kp= 1

LENGTH TO BE RESTRAINED AT 210 PSI FOR GRANULAR BACKFILL, L (ft.)

D (in.)	Bc (in.)	θ (deg)	Depth of Fill to Top of Pipe (ft.) H = 6	7	8	9	10
84	96.5	90.0	L = 102.7	99.4	96.3	93.4	90.7
84	96.5	73.0	L = 82.2	79.9	77.7	75.6	73.6
84	96.5	55.0	L = 62.1	60.6	59.1	57.8	56.5
84	96.5	38.0	L = 43.7	42.8	41.9	41.0	40.2
84	96.5	10.0	L = 12.2	12.0	11.9	11.7	11.5
84	99	90.0	L = 100.0	96.8	93.8	91.0	88.3
84	99	73.0	L = 80.0	77.8	75.6	73.6	71.7
84	99	55.0	L = 60.5	59.0	57.6	56.2	55.0
84	99	38.0	L = 42.5	41.6	40.8	40.0	39.2
84	99	10.0	L = 11.9	11.7	11.6	11.4	11.2

Note: D = I.D. of pipe

Condition II. Groundwater at Ground Surface (During Heavy Flood)

Maximum Sustained Pressure, P = 210 psi

Factor of Safety, Sf = 1.25

Trench Backfill Parameters:

$r_b' =$ 60 pcf
Øb= 25 degree
Cb= 0 psf
δ = 0.75 Øb degree
f = 0.5 Cb psf

In-Situ Soil Parameters:

$r_s' =$ 62.5 pcf
Øs= 0 degree
Cs= 2000 psf
Kp= 1

LENGTH TO BE RESTRAINED AT 210 PSI FOR GRANULAR BACKFILL, L (ft.)

D (in.)	Bc (in.)	θ (deg)	Depth of Fill to Top of Pipe (ft.) H = 6	7	8	9	10
84	96.5	90.0	L = 128.1	123.0	118.3	114.0	109.9
84	96.5	73.0	L = 99.5	96.1	92.9	90.0	87.2
84	96.5	55.0	L = 73.0	70.9	69.0	67.1	65.3
84	96.5	38.0	L = 50.0	48.9	47.7	46.6	45.6
84	96.5	10.0	L = 13.4	13.2	13.0	12.8	12.6
84	99	90.0	L = 124.7	119.8	115.2	111.0	107.0
84	99	73.0	L = 96.8	93.5	90.5	87.6	84.9
84	99	55.0	L = 71.1	69.1	67.1	65.3	63.6
84	99	38.0	L = 48.7	47.6	46.5	45.4	44.4
84	99	10.0	L = 13.1	12.9	12.7	12.5	12.3

Note: D = I.D. of pipe

SAMPLE CALCULATION FOR LENGTH TO BE RESTRAINED

For a 90 degree horizontal bend in a 84 in. I.D. pipe (O.D. 96.5 in.) using surge pressure of 210 psi.
The pipe is covered with 8 feet of granular backfill.
The depth to bottom of trench is about 16.5 feet.
The groundwater level is 6 ft. below ground surface.

Design Parameters are:

$P = 210$ psi
$D = 84$ inches
$Bc = 96.5$ inches $= 8.04$ feet
$\theta = 90$ degree
$H = 8$ feet
$Sf = 1.25$

Backfill:		In-situ soil:	
$r_b =$	120 pcf (0 to 6 ft.)	$r_s =$	125 pcf (0 to 6 ft.)
$r_b' =$	60 pcf (below 6 ft.)	$r_s' =$	62.5 pcf (below 6 ft.)
$\phi b =$	25 degree	$\phi s =$	0 degree
$Cb =$	0 psf	$Cs =$	2000 psf
$R =$	2/3		

and $K = 4\tan\theta/2 = 4 \times \tan(90^\circ/2) = 4.000$

$A = \pi D^2/4 = \pi \times (84^2/4) = 5541.8 \ (\text{in.}^2)$

$Ap = \pi Bc/2 = \pi \times (8.04/2) = 12.63 \ (\text{ft}^2/\text{lin ft.})$

$f = 0.5\ Cb = 0.5 \times 0 = 0$ psf

$W = r_b.H.\pi.Bc.R = (120\times6+60\times(8-6)) \times \pi \times 8.04 \times (2/3)$
$= 14147.6$ (lbs/lin ft.)

$\delta = 0.75\ \phi b = 0.75 \times 25^\circ = 18.75$ (degree)

$Fs = Ap.f+W.\tan\delta = 12.63 \times 0 + 14147.6 \times \tan(18^\circ.75)$
$= 4802.5$ (lbs/lin ft.)

$Hc = H+Bc/2 = 8 + (8.04/2) = 12.02$ (feet)

$Kp = \tan^2(45^\circ+0s/2) = \tan^2(45^\circ + 0^\circ/2) = 1$

$Pp = r_s.Hc.kp + 2.Cs.\sqrt{Kp}$
$= (125 \times 6 + 62.5 \times (12.02-6)) \times 1 + 2 \times 2000 \times \sqrt{1}$
$=5126.3$ (psf)

find $$L = \frac{Sf.K.P.A}{k.Fs.+Bc.Pp} = \frac{1.25 \times 4.0 \times 210 \times 5541.8}{4.0 \times 4802.5 + 8.04 \times 5126.3}$$

$= 96.3$ (feet)
